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AEROSPACE
AIR TRANSPORT
HIGHWAY
PIPELINE
URBAN TRANSPORTATION

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CONTENTS

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DISCUSSION

Proc. Paper 16359

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[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

16398 HIGHWAY SAFETY APPURTENANCES

KEY WORDS: Breakaway signal supports; Break-away-signs; Breakaway sign supports; **Breakaway structures; Highway safety; Highway safety act of 1966;** Highway safety standards; Highway signs; **Safety device effectiveness; Safety engineering; Safety factors;** Safety standards; State of the art studies; **Traffic safety**

ABSTRACT: To reduce the horrendous toll on our nation's roadways, (50,000 fatalities per year) highway agencies, researchers, and industries are responding effectively to safety legislation by Congress. Among the many safety initiatives taken was the systematic installation of forgiving appurtenances developed to reduce the 17,000 - 22,000 deaths involved in errant vehicle collisions. Statistical data, for errant vehicle roadway departures and for accident-frequency collisions with the existing roadside obstacles, define the enormity of the problem confronted. Traffic railings and impact attenuators are designed as containment or redirective protective safety appurtenances, or both as the situation requires. Devices are explored that could be installed along our highways and used as temporary systems to protect motorists and workmen during construction and maintenance operations.

REFERENCE: Tamanini, Flory J. (Research Consultant, Energy Absorption Systems, Inc., 1104 Vassar Road, Alexandria, Va. 22314), "Highway Safety Appurtenances: State-of-the-Art," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16398**, July, 1981, pp. 385-399

16389 VALIDATING AIRPORT SIMULATION MODELS

KEY WORDS: **Airports;** Air traffic; **Air transportation;** Computer applications; **Computerized simulation;** Data analysis; Logistics; **Models; Model studies;** Sensitivity analysis; **Simulation models;** Statistical data; **Validation**

ABSTRACT: A procedure is presented for validating an airport simulation model. The emphasis is on basic principles of validation and inherent problems in comparing simulation model estimates with observable real-world data. The validation procedure consists of three major steps: (1) An evaluation of model logic, assumptions, inputs, and outputs; (2) a sensitivity analysis of the model estimates; and (3) a comparison of model estimates with observed data. An examination is presented of two important statistical considerations for simulation models: auto-correlation and model convergence. Guidelines are given for interpreting the results of statistical hypothesis tests.

REFERENCE: Dunlay, William J., Jr. (Sr. Consultant, Peat, Marwick, Mitchell & Co., P.O. Box 8007, San Francisco, Calif. 94128), "Simulation Model Validation: Airport Applications," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16389**, July, 1981, pp. 401-412

16386 WINDMILLS AND WIND CHARACTERISTICS

KEY WORDS: **Aerodynamic characteristics; Climatology;** Electric power generation; Numerical calculations; Simulation; **Site evaluation;** Site investigation; **Site selection;** Wind forces; **Wind (meteorology); Windmills; Wind power generation;** Wind tunnel models

ABSTRACT: Past experience with power generation by windmills indicates that the most important factor controlling success or failure is site wind characteristics. Incorrect placement on a site sheltered by buildings, terrain or agricultural growth may drop performance to one-third of the original expectations. Conversely, the appropriate hill or ridge shape may amplify power available. Site selection procedures, which are the "pick and shovel" of the modern wind prospector, include statistical climatology, numerical simulation, and physical simulation in meteorological wind tunnels. Laboratory measurements of wind overspeed, streamline patterns, and turbulence changes over idealized topography are compared with "frozen vorticity" numerical models. Field measurement of wind velocity and direction over the Rakaia Gorge region, Southern Alps, New Zealand, compare favorably with wind tunnel measurement over a 1/5000 scale model.

REFERENCE: Meroney, Robert N. (Prof., Fluid Mechanics and Wind Engrg. Program, Colorado State Univ., Fort Collins, Colo. 80523), "Prospecting for Wind: Windmills and Wind Characteristics," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16386**, July, 1981, pp. 413-426

16393 EMPIRICAL MODELS OF TRANSIT SERVICE AREAS

KEY WORDS: **Accessibility; Access routes;** Buses; Bus usage; Commuting patterns; Distance; Railroads; Subways; **Time factors; Transit systems; Transportation planning; Transportation studies;** Urban buses; **Urban transportation**

ABSTRACT: Empirical tools for planning access to transit systems are developed. Transit access is defined as those portions of the journey spent getting to the transit system, and then from the transit system to the destination point. The usual transit modes include the following: walking, park-and-ride, kiss-and-ride, paratransit feeder, transit feeder, taxi and bicycle. The impact of the access characteristics on transit ridership and the normative definition of the area accessed by the transit system are explored. The models presented concern bus routes (in Vancouver, St. Louis and Washington, D. C.) and commuter rail and express bus service (in northeastern New Jersey). The models determine normative classifications for transit service areas on the basis of access distance and time.

REFERENCE: Lutin, Jerome M. (Mgr., Regional Transportation Planning Dept., Parsons Brinckerhoff, Quade & Douglass, Inc., One Penn Plaza, New York N.Y. 10119), Liotine, Matthew, and Ash, Thomas M., "Empirical Models of Transit Service Areas," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16393**, July, 1981, pp. 427-444

16392 SATELLITE DATA FOR WATER MANAGEMENT

KEY WORDS: Benefit cost analysis; **Data collection systems; Economic analysis;** Flood control; **Hydrologic data;** Hydrology; Real time operations; **Satellites;** Sensors; **Water management (applied); Water resources**

ABSTRACT: The U.S. Geological Survey is utilizing satellite data relay to automate the timely acquisition and distribution of hydrologic data from more than 200 remote data-collection sites. This activity has been part of a nationwide study program carried out for more than 8 years. In that period, satellite telemetry has proven to be reponsive, flexible, reliable and, when compared with other telemetry systems, cost effective. The use of satellites to telemeter data from remote locations results in the following: real-time hydrologic data being made available to water-data users, reduced site visits for manual retrieval of data, and the ability of the data collector to monitor instrumentation operation and hydrologic conditions at unmanned sites. A Geological Survey economic study projecting the availability of real-time data for flood warning and irrigation water management forecasts that the accumulated benefits derived from use of those data outweigh the added costs of providing the telemetry system .

REFERENCE: Shope, William G. (Chf., Data Relay Project, Water Resources Div., United States Geological Survey, Reston, Va. 22092), and Paulson, Richard W., "Data Collection via Satellite for Water Management," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16392**, July, 1981, pp. 445-455

16401 ANALYSIS OF WATER DISTRIBUTION SYSTEMS

KEY WORDS: **Fluid flow;** Hydraulics; **Network analysis;** Network analysis theory; Networks; **Nodes;** Optimization; **Pipe flow;** Water consumption; Water demand; **Water distribution; Water transportation**

ABSTRACT: Various techniques presently available for the analysis of water distribution systems, called node head analysis (NHA) techniques, analyze the distribution systems, assuming that the nodal demands can be satisfied by providing additional source heads or boosting pressures, if necessary. When such additional heads or pressures are not available, some consumption nodes fail, partially or completely, to satisfy the nodal demands. To locate such nodes and to estimate the actual nodal supplies, an approach termed node flow analysis (NFA) is developed herein. The NFA problem is considered a flow optimization problem with linear and nonlinear constraints, some being alternate constraints. The solution approach is based on repetitive NHA of the distribution system. The necessary theory for NFA is described, and the procedure is developed and illustrated through a design example.

REFERENCE: Bhave, Pramod R. (Prof.), "Node Flow Analysis Distribution Systems," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16401**, July, 1981, pp. 457-467

16403 BURIED PIPELINES AND LOAD TRANSFER

KEY WORDS: **Buried pipes; Contact pressure;** Earth pressure; **Load transfer;** Pipe joints; **Pipelines; Pipeline transportation;** Settlement (structural); Stresses; Subsurface structures; **Underground structures**

ABSTRACT: Flexible jointed buried pipelines, when subject to conditions that result in differential settlements, transfer loads along the pipeline which in turn modify contact pressures and settlements for each length of pipe. Angular distortion between the individual lengths of pipe thus results in opening joints and creating a potential for leakage. Expressions are derived, based on certain assumptions, that permit the determination of the transfer loads, as well as the resulting contact pressures, pipe settlements, and angular distortions between individual pipe sections adjacent to structures. Consideration is given in the derivations to the transitional loading on pipe which is a consequence of its support at a structure. The procedures for load transfer determinations are also applied to pipelines subject to concentrated surface loadings. Additionally, the theory is extended to the case of buried pipelines having horizontal unbalanced thrust at fittings.

REFERENCE: Scarino, John H. (Principal Associate, Clinton Bogert Associates, Consulting Engineers, 2125 Center Ave., Fort Lee, N.J. 07024), "Buried Pipelines: Settlement Modification and Load Transfer," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16403**, July, 1981, pp. 469-486

16397 PIPELINE CORRIDORS IN DEVELOPING AREAS

KEY WORDS: **Aqueducts;** Encroachment; History; Land use; **Pipelines; Public utilities; Right of way (land);** Time factors; **Urban development;** Urban planning; Water management (applied); **Water pipelines; Water transportation**

ABSTRACT: The San Diego County Water Authority is a wholesale agency providing water to the approximately 1,700,000 residents of San Diego County through its 24 member agencies. It owns, operates and maintains approximately 210 miles (338 km) of aqueduct pipelines. Original construction of the pipelines was in locations remote from urban development. However, during the past 20 years, urban development has encroached to the rights-of-way and, in many cases, over the rights-of-way. Several unique arrangements for the joint use of the rights-of-way corridors with other public agencies have provided benefits to the general public. Some of the problems the Authority has encountered are explained, and recommendations are presented on how similar problems can be avoided by others.

REFERENCE: Ogden, Buckley L. (Asst. Chf. Engr., San Diego County Water Authority, 2750 Fourth Ave., San Diego, Calif. 92103), "Pipeline Corridors in Developing Areas," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16397**, July, 1981, pp. 487-495

16405 REHABILITATION OF SANITARY SEWER PIPELINES

KEY WORDS: **Corrosion; Dewatering;** Environmental engineering; Ground water; **Grouting;** Infiltration (water); **Pipeline transportation;** Plastic pipes; **Rehabilitation;** Sanitary engineering; **Sanitary sewers; Sewer pipes**

ABSTRACT: An extensive investigation into the methods and materials available for use in rehabilitating sanitary sewers is presented. Information has been developed on the features and benefits of both internal and external sewer pipeline rehabilitation. The analysis of various types of failures that occur in sanitary sewer pipelines shows that a variety of correction methods can be utilized. Twenty-three problem areas for rehabilitation have been identified, along with a choice of correction methods and special considerations for them. Dewatering of the existing system for rehabilitation is an important determinant of the method and material selected. In addition to dewatering, the diameter, cross section, variation inflow, number connections, access, location, depth and nature of effluent, movement of groundwater and soil settlement conditions are all important factors of rehabilitation evaluation.

REFERENCE: Ouellette, Herve (Consultant, 919 Oriole Dr., Laguna Beach, Calif. 92651), and Schrock, B. Jay, "Rehabilitation of Sanitary Sewer Pipelines," *Transportation Engineering Journal,* ASCE, Vol. 107, No. TE4, **Proc. Paper 16405**, July, 1981, pp. 497-513

U.S. Customary-SI Conversion Factors

In accordance with the October, 1970 action of the ASCE Board of Direction, which stated that all publications of the Society should list all measurements in both U.S. Customary and SI (International System) units, the following list contains conversion factors to enable readers to compute the SI unit values of measurements. A complete guide to the SI system and its use has been published by the American Society for Testing and Materials. Copies of this publication (ASTM E-380) can be purchased from ASCE at a price of $3.00 each; orders must be prepaid.

All authors of *Journal* papers are being asked to prepare their papers in this dual-unit format. To provide preliminary assistance to authors, the following list of conversion factors and guides are recommended by the ASCE Committee on Metrication.

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
square inches (sq in.)	square millimeters (mm^2)	645
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
cubic inches (cu in.)	cubic millimeters (mm^3)	16,400
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb) mass	kilograms (kg)	0.453
tons (ton) mass	kilograms (kg)	907
pound force (lbf)	newtons (N)	4.45
kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	pascals (Pa)	47.9
pounds per square inch (psi)	kilopascals (kPa)	6.89
U.S. gallons (gal)	liters (L)	3.79
acre-feet (acre-ft)	cubic meters (m^3)	1,233

Highway Safety Appurtenances: State-of-the-Art[a]

By Flory J. Tamanini,[1] F. ASCE

(Reviewed by the Highway Division)

Introduction

Beginning in the mid-1960s, highway officials exhibited considerable concern about the toll in human lives and suffering associated with highway accidents. With enactment of Congressional legislation in 1966 (3), the problem of highway safety was officially acknowledged as serious on a nationwide basis. Vigorous programs were initiated by the Federal and state governments and the private sector alike to define and understand the "highway safety" problem. Conferences and seminars were convened involving not only specialized technologists but also more importantly multidiscipline groups. From these endeavors, the magnitude of the problem was confirmed in light of 1966 accident statistics (1), i.e.: (1) Fatalities—53,000; (2) injuries—1,200,000; and (3) estimated Societal Cost of $10,000,000,000.

Accident statistics vary widely from state to state. Yet it has been reported that on a national basis, single-vehicle ran-off-the-road collisions with roadside obstacles constitute 33%–40% of the total annual motor vehicle fatalities (1,16). This figure represented 17,000–22,000 deaths, a tremendous toll.

Responding to passage of the 1966 Highway Safety Act, the Administration took prompt, decisive action to show prioritization of highway safety. In 1967, the Federal Highway Administration (FHWA) launched a short-term, quick-payoff research program (27) which provided an effective technological stimulant for subsequent research by states and the private sector.

In the mid-1960s, there were already several research endeavors underway by some state agencies, the private sector, and entrepreneurs to develop safety appurtenances designed to reduce the severity of accidents. Several of these are worthy of note. The New York State Department of Public Works developed an effective, strong box-beam (weak post) traffic railing (13). This system was installed immediately upon completion of successful testing by numerous states

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Note.—Discussion open until December 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on August 1, 1980. This paper is part of the Transportation Engineering Journal of ASCE, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. TE4, July, 1981. ISSN 0569-7891/81/0004-0385/$01.00.

which had serious traffic railing needs. The Texas Highway Department initiated a research and development study of a variety of sign supports in an effort to minimize personal injury and property damage when impacted by out-of-control vehicles. Following this initial effort, 14 states in cooperation with FHWA sponsored a 3-yr research endeavor which resulted in safety design criteria for breakaway metal sign support structures (5). The breakaway (or slip-base) technique was soon applied most effectively to tall roadside luminaire supports. The development and adaptation of the New Jersey shaped concrete median barrier, complemented by the General Motors version, proved to be a most effective concept to deter median cross-overs by errant vehicles (8,9). Its real value however was to be appreciated in later years.

Errant Vehicle Impact Conditions

Generally speaking, the following conditions can be said to exist when an errant vehicle departs a roadway (17,26). (For specific locations, site-specific environmental and accident-frequency data should be consulted.): (1) 80%–85% depart within 0°–30° of the highway axis; and (2) 70%–80% of the errant vehicles

TABLE 1.—Frequency of Struck Roadside Objects

Object or obstacle (1)	Reported Accidents, as a Percentage		
	Fatal (2)	Injury (3)	Total (4)
Trees	21.5	14.2	9.0
Traffic railings	16.9	14.1	13.3
Utility poles	15.9	20.6	11.2
Bridge piers, abutments	7.3	2.2	1.5
Traffic signs, signals	4.5	3.4	4.0

strike a fixed object within 30 ft of the edge of the roadway.

The tabulation found in Table 1 was compiled from the 1971–72 highway accident statistics submitted to the FHWA by Kansas, Pennsylvania, Massachusetts, and Michigan. The data are concerned with the most frequently struck roadside object by an errant vehicle. (Total accidents—1,281,000; fatalities—2,959; and injuries—87,502).

Design Requirements

Design requirements for roadside appurtenances to facilitate safe impacts by vehicles in the 2,000 lb–5,000 lb (907 kg–2,267 kg) weight range have been consistently specified in research sponsored by the FHWA, the state, and the private sector. Acceptable deceleration of 12g, averaged over the highest 50 msec of impact, is the specified criterion with a maximum acceptable onset rate not to exceed 500g/s. The Transportation Research Circular Number 191 (20) specifies performance criteria for the acceptability of longitudinal barriers, crash cushions, and breakaway or yielding supports. Vehicle impact speeds

of 20 mph–60 mph (32 km/h–97 km/h) for vehicles weighing 2,250 lb–4,500 lb (1,020 kg–2,040 kg) are stated. Vehicle impact angles ranging from 0°–25° are specified depending on the appurtenance being tested.

In light of the ever-increasing number of smaller and lighter weight cars entering the traffic stream in recent years, there has been an increase in emphasis and priority to accommodate these lighter and normally smaller vehicles. A liberal interpretation of industrial production figures of passenger vehicles since 1970 discloses that the annual United States registration of compact, subcompact, and sports cars has increasingly ranged from approx 32%–46% of all new car purchases (18). More detailed information on vehicle size and weight distribution can be obtained from the National Highway Traffic Safety Administration (NHTSA) or industrial automobile production records.

A FHWA Bulletin issued on April 26, 1979, establishes a lower weight and size value to the existing design criteria for highway safety appurtenances (12). The mini-sized car weighing about 1,700 lb–1,800 lb (770 kg–815 kg) must now be considered. In addition, the existing criteria states that school buses, intercity buses, and even tractor-trailer vehicles will be included in research and development studies.

According to the American Association of State Highway and Transportation Officials (AASHTO) design specifications (25):

> Satisfactory dynamic performance is indicated when the maximum change in momentum for a standard 2,250-lb (1,020-kg) vehicle, or its equivalent, striking a breakaway support at speeds from 20 mph–60 mph (32 km/h–97 km/h), does not exceed 1,100 lb-sec (4,893 N-s), but desirably does not exceed 750 lb-sec (3,336 N-s).

Highway Safety Appurtenances

Luminaire Supports.—Adequate lighting of our highways and streets has long been acknowledged as a necessity to provide vehicle drivers with needed visibility. This is being accomplished by the use of headlights and roadside luminaires. Prior to the development of the breakaway or slip-type luminaire supports, it was estimated that approx 1,300 lives were lost annually in vehicle collisions with such appurtenances. Exact figures on a nationwide basis have been impossible to obtain. Through implementation of conducted research by FHWA and States, vehicle collisions with luminaire supports have been reduced to "fender-bender" status. Fatalities in such occurrences have become almost nonexistent.

It is a Federal requirement that luminaire supports must be located no closer than 30 ft (9.1 m) from the edge of the roadway. If this cannot be done, they must be made fail-safe.

In light of the 1975 AASHTO requirement (25) which highlighted small-car safety considerations, many manufacturers of luminaire supports had to requalify their products.

Typical luminaire supports in the 30 ft–55-ft (9.1-m–16.8-m) height range which have been qualified as acceptable fall generally within the following types: (1) The shoe-mounted type, aluminum or steel; (2) the progressive-shear type, steel; (3) the transformer base, aluminum; (4) shoe-base with riser, aluminum; (5)

notched steel bolt insert, high-strength steel; and (6) fluted base supports, aluminum.

Specific makes and models are approved by the FHWA Division Office in each state.

Utility Poles.—There are about 88,000,000 utility poles installed along our highways, roads, and streets, annually taking a tragic toll of lives. It is hoped that effective countermeasures can soon be available for new installations and for retro-fitting existing utility poles as a result of ongoing research.

It has been estimated in Government-sponsored research reports that approx 2,700 lives are lost each year in automobile collisions with utility poles. These are normally 10 in.–12 in. (250 mm–300 mm) in diameter at the base and are located in most cases within several feet of the traveled way, a most tragic roadside deathtrap. The FHWA is currently sponsoring a comprehensive research

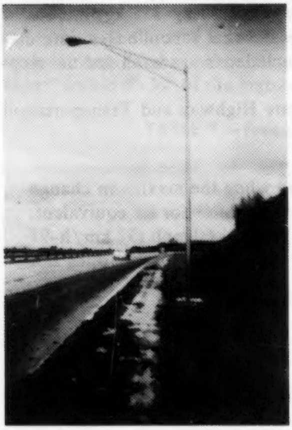

FIG. 1.—Typical Roadside Luminaire Support

FIG. 2.—Utility Pole on Rural Road

and development study at the Southwest Research Institute (SWRI) (6). This is a new endeavor. The Lincoln Electric System has also sponsored a research study at the University of Nebraska (19). Results from research conducted thus far indicate that safe drive-away vehicle collisions with utility poles may very well be possible, and still maintain the overhead wires in place. Various fail-safe concepts are being evaluated.

Roadside Sign Supports.—Soon after the successful development of breakaway and slip-base luminaire supports, the technology was transferred effectively to shoulder-mounted, roadside sign supports. The FHWA and 14 states funded an accelerated research and development endeavor at the Texas Transportation Institute (TTI) to accomplish this technology transfer to a second adaptation. This cooperative pursuit was so successful that the resulting fail-safe aspects were mandated to be implemented by the FHWA on all Federal-Aid highways

in states prior to preparation of the final research report (5). While both "frangible" and "slip" base concepts were proved acceptable, the latter type became more preferred for implementation.

State agencies and private industry became encouraged to pursue the development of new competitive systems for initial installations and for retrofitting existing structures. Prominent of the many adaptations and new types was the ingenious Break-Safe Sign Support Base by Transpo-Safety Systems for use on single or multiple-post sign support structures and for retrofit use as well as for new construction.

Some State highway agencies have stated informally that 70%–90% of impacts with sign and luminaire supports go unreported—in other words, they are drive-away fender-benders. This is an indication of achieved "increased survivability" in vehicle accidents on our highways and streets.

For the decade 1965–1975, attention was focused on the design of highway signposts to be breakaway for large automobiles. However, recent studies sponsored by the FHWA on the performance of smaller sign posts during impact

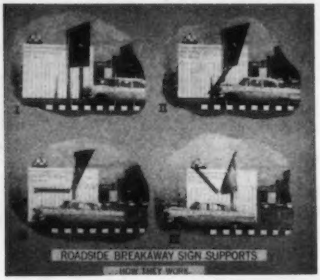

FIG. 3.—Roadside Breakaway Sign Supports—How They Work

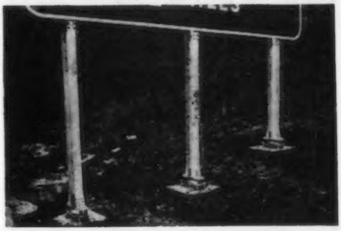

FIG. 4.—New BREAK-SAFE Sign Support

by small vehicles disclosed a serious oversight. Small-car impacts at moderate to high speeds resulted in disastrous performance for single signposts previously believed to be safe structures. This new study was conducted by the TTI with the excellent cooperation of many fabricators of small signposts. Four separate technical reports (21,22,23, and 14) covering this FHWA-sponsored research have been published.

Protection in Median.—Errant vehicles which traverse the median frequently result in violent head-on collisions with either oncoming vehicles or with highway sign-posts, luminaire supports, or median bridge piers. It has been estimated that the annual fatality rate is about 7% of the single-vehicle fatalities, or 1,400 deaths occurring in median collisions. The worst collisions are those involving small cars in head-on situations or where protective countermeasures have not been provided.

On our multilane, separated highways there are a variety of protective median barriers being employed today in high accident-potential locations, particularly

where accident experience warrants it or where the median width is less than 30 ft (9.1 m). Traffic railings, such as median barriers, guardrails, and bridge rails, must satisfy the provisions of FHWA/AASHTO specifications. To qualify for use on Federal-Aid highway projects, prospective median barriers must safely accommodate the stipulated head-on and angle impacts by both small and large-size cars. A 1977 AASHTO publication, identifies Federally approved systems. (15)

Blocked-out, double-faced steel guardrails on either wood or steel posts probably constitute the most frequently used traffic railing. An improved version of railing known as Thrie Beam, or Tri-Gard, has recently become popular in construction of bridge rail retrofit safety-improvement projects. Another steel median barrier which has proven to be quite effective is the New York Box Beam Median Barrier. This system has excellent characteristics for safely attenuating and redirecting impacting small and large-size cars.

Being greatly concerned with the importance of median barrier technology, the FHWA together with 21 interested state highway agencies sponsored a 3-yr research and development study at the SWRI. A comprehensive study was undertaken of the design, construction, and performance of a variety of shapes

FIG. 5.—Typical Unprotected Bridge Pier in Narrow Median

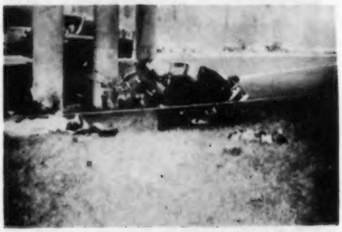

FIG. 6.—Disastrous Incident at Unprotected Median Bridge Pier

for the concrete median barrier (CMB). The resulting final report is an excellent technical compendium (9). The shape of the CMB's is designed to redirect colliding vehicles with minimum damage and personal injury while preventing crossover accidents with oncoming traffic. There are approx 3,600 miles (5,800 km) of the CMB on our nation's highways. Hundreds of thousands of automobile tire skid marks seen on the CMB's attest to the ability of this barrier in preventing cross-over, head-on collisions.

The aluminum industry provides states with a special traffic railing set of components known as the Semi-Elliptical Railing System which can be assembled in various configurations to serve as guardrail, median barrier, or bridge railing. This industry has also made available two lightweight median barrier designs of extruded aluminum components: (1) The DOW-type; and (2) the MAGNODE-type. Their popularity for use on existing bridges is due to their lightweight frequently requiring no additional structural support and recognized safety performance.

Hazardous ends of median barriers must be flaired away from adjacent traffic lanes at least 30 ft (48 m). Where appropriate flairs cannot be incorporated

in the median barrier design, impact attenuators are being utilized most effectively with excellent results. States are using two types of inertial barriers: (1) ENERGITE and FITCH MODULES; and (2) redirective-type attenuators: HI-DRO, HI-DRI, and the G-R-E-A-T Systems.

ENERGITE and FITCH MODULES are 36-in. (0.91-m) diam, 36-in. (0.91-m) high closed cylinders fabricated of frangible plastic. They contain varying amounts of sand, 400 lb–2,100 lb (182 kg–953 kg), depending on their locations in the attenuator array.

The HI-DRO System is 40 in. (0.94 m) high, is comprised of water-filled tubes with expulsion valves at their tops activated by an impacting vehicle.

FIG. 7.—G-R-E-A-T Attenuator at Blunt-End CMB

FIG. 8.—Provision of Safe Attenuation and Redirection by G-R-E-A-T Units (Note Thrie-Beam/Tri-Gard Railing)

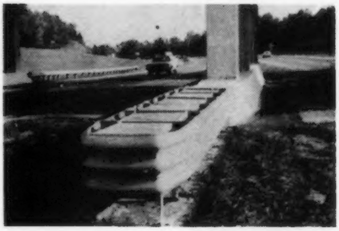

FIG. 9.—G-R-E-A-T Installation at Median Bridge Pier

FIG. 10.—FITCH Barrier Protection at Exit Ramp Bridge Pier

Overlapping plastic side panels provide for vehicle redirection for side impact collisions.

The HI-DRI System is similar dimensionally to the HI-DRO attenuator. Instead of water-filled tubes, the system utilizes frangible cells of vermiculite concrete assembled in waterproof cartridges. The cells are individually waterproofed and wrapped with wire to provide for desired deceleration upon application of a dynamic force. Vermiculite concrete has a density of approx 35 lb/ft^3 (560 kg/m^3).

The G-R-E-A-T System utilizes vermiculite cartridges as the compression medium for safely containing impacting vehicles in a head-on situation. It has

a frangible curved plastic nose and overlapping steel panels (Thrie Beam or Tri-Gard) to provide safe redirection to vehicles in side angle impacts.

An untold number of drive-away impacts by small and large automobiles are known to have involved these lifesaving systems.

Not only are these impact attenuators providing lifesaving protection for motorists at the ends of traffic railings, but they are also providing effective protection when installed directly at median bridge piers.

Protection at Exit Ramps.—Where hazardous roadside obstacles cannot be removed from the exit ramp, effective utilization of impact attenuators has been resorted to by the FHWA and the states in new construction and site-improvement projects at selected locations.

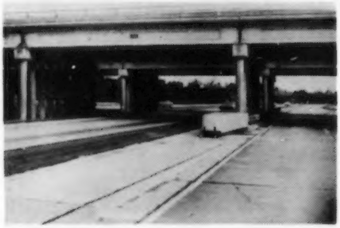

FIG. 11.—HI-DRO Crash Cushion at Bridge Pier in Narrow Median

FIG. 12.—Dangerous Elevated Gore—Before Accidents and Installation of Countermeasure

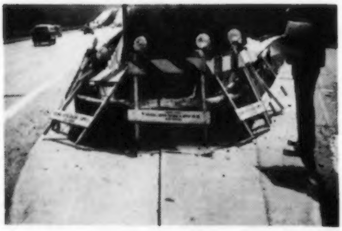

FIG. 13.—Evidence of Vaulting Vehicle Accident at Elevated Gore

FIG. 14.—HI-DRO Attenuator Solution for Elevated Gore Problem

Both inertial-type and redirective-type crash cushions have established a record for saving many lives and for minimizing injury and vehicle damage. Many states have published evaluation reports on their effectiveness and cost-benefits (2,10,11,31, and 33). Both types of attenuators have life-saving capability. At locations where impact incidence is frequent or where moderate-angle hits are anticipated, redirective type attenuators are preferred because of their effectiveness and lower maintenance costs.

Forgiving Bridge Rails.—Tremendous strides have been made in the past 5 yr in the research and development of much safer bridge rails than those built in the pre-1970 era. In past years, bridge rails were designed to stop vehicle

penetration utilizing a 10,000 lb static load design specification. Many existing types do not accomplish this objective. Research studies initiated by the FHWA and AASHTO/TRB were most successful in developing bridge rails that not only prevented penetration by small and larger-size cars, school buses, and intercity buses, but also provided safe redirection for this wide range of vehicle sizes.

A redirective-type bridge railing using the New York Box-Beam blocked out by frangible aluminum tubes spaced on about 6-ft (1.8-m) centers was developed from one FHWA study at the SWRI. This system was designed to provide adequate attenuation and redirection for both small and large size passenger cars. Only two installations of this system are known to have been made: (1)

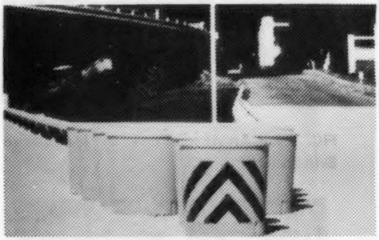

FIG. 15.—ENERGITE Crash Cushion Solution for Elevated Gore Problem

FIG. 16.—FITCH Crash Cushion After Driveaway Vehicle Impact

FIG. 17.—HI-DRI Attenuator at Butterfly Gore Sign

In Connecticut; and (2) another in Virginia.

Following this research, the SWRI was also successful in completing another FHWA-sponsored study dealing with the technology to retrofit existing bridge rails. The comprehensive work resulting in a large variety of retrofit railing types is contained in the final report (32).

Another productive FHWA-sponsored research study was conducted at the SWRI. It resulted in the development of a successful collapsing-ring bridge rail system. While this system had as its objective the safe accommodation of small and large passenger cars, it finally wound up with most unusual results. The system as finally developed, was also capable of safely redirecting school

buses and intercity buses. Indeed a spectacular outcome. The final report presents a comprehensive coverage of the study (7).

Having accomplished noted effective results from its sponsored bridge rail research, the FHWA has recently sponsored a research study to develop the "ideal" traffic barrier. The objective is the development of a "self-restoring" longitudinal barrier system known by the acronym, SERB—self-restoring barrier.

FIG. 18.—Ineffective Bridge Rail

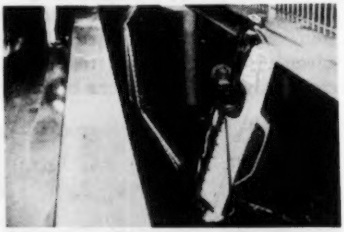

FIG. 19.—Another Type of Ineffective Bridge Railing

FIG. 20.—Forgiving and Redirective Bridge Rail—Frangible Aluminum Type

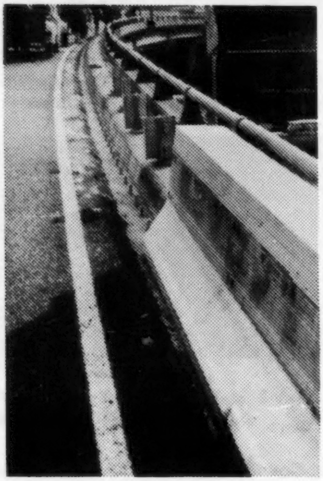

FIG. 21.—Bridge Rail Retrofit—Aluminum Barrier Shape Type

The SERB is a very simple longitudinal barrier designed to redirect impacting passenger cars and school buses. It utilizes two wide flex-beam guardrail sections (Thrie-Beam or Tri-Gard), 20-in. deep, welded back to back. The fabricated tritubular-looking railing is then fastened to the steel posts by a hinged connection. Through this inertial system, the kinetic energy of an impacting vehicle is absorbed

while being safely redirected. Damage to the colliding vehicles and the barrier system was observed to be minor.

Construction Zone Safety.—Technical literature and periodicals have contributed magnificently in the technology transfer of this rapidly developing state-of-the-art. Providing safety for both work crews and motorists in the construction zone has been an FHWA-specified, high-priority, emphasis area.

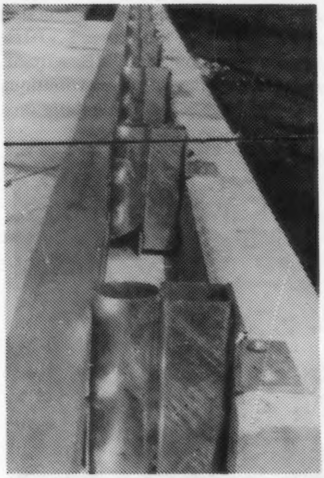

FIG. 22.—Another Bridge Rail Retrofit

FIG. 23.—Collapsing Steel Ring Bridge Rail

FIG. 24.—G-R-E-A-T Attenuator—Protection at Dangerous Bridge Rail End

FIG. 25.—CZ G-R-E-A-T Protection for Motorists and Work Crews

In 1976, the FHWA issued instructions to replace dangerous wood and hazardous barricades with safer systems having structural integrity and the capability to redirect errant vehicles back onto the highway. To promote its "safety in construction zones" effort, the FHWA issued three excellent slide-tape packages on the use of temporary barriers, barricades, and signing and pavement

marking (4,24,30). These were made available on a nationwide basis. The private sector working with the states almost immediately responded by producing elemental, redirective traffic railings. These included steel I-beam railings and short, movable 10 ft–20-ft connectible concrete median barriers. For protecting occupants of vehicles impacting the ends of traffic railings, portable impact attenuators were found to be most adaptable and efficient. Notable among these commercially available solutions were the inertial and frangible barrier types and the more sophisticated Guardrail Energy Absorbing Terminal for Construction Zones (the G-R-E-A-T cz).

Temporary barriers of the CMB shape, crash cushions for CMB termini, and truck-mounted attenuators (TMA's) are making noteworthy contributions in the furtherance of safety.

FIG. 26.—Truck-mounted Attenuator at Work Site

FIG. 27.—HI-DRO Cell Cluster at Automatic RR X-ing Gate

The Construction Zone G-R-E-A-T provides considerable protection at a variety of construction sites. It offers the same response and performance characteristics as the standard G-R-E-A-T System mentioned previously.

Railroad/Highway Grade Crossing.—Annually about 1,000 lives are lost due to accidents at railroad-highway grade crossings. They are the most tragic types of collisions. Frequently school buses are involved with disastrously high death tolls.

In cooperation with railroad companies, FHWA, state, and local governments are making great progress in improving the environment and traffic control devices at crossings. Federal funding has been made available not only for improving crossing sites, and even eliminating them by the construction of grade separations, but also by providing protection for motorists and for signals and

automatic gate structures. These devices provide survivability to occupants of both large and small cars in moderate to high-speed impacts.

Protection at Tollgate Facilities.—For decades, concrete monstrosities were constructed in front of the toll road collectors' booths for their occupants' protection. In recent years however, with the advent of concern for motorists, crash cushion technology has been adapted to these roadside obstacles. Either in new construction or in retrofitting the existing concrete hazardous structure, hydraulic impact attenuators are being utilized as an enhancement to highway safety. These water-filled tube crash cushions are known as HI-DRO Cell Clusters. They are capable of safely attenuating moderate-speed impacts of about 45 mph (72 km/h) of small and large automobiles, resulting in minor or no injury and minimum vehicle damage.

Motor Vehicle Accident Statistics, 1979

The following information appears in "Accident Facts—1980 Edition," an annual publication of the National Safety Council. The data portrays a slight

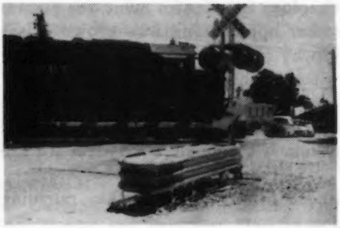

FIG. 28.—G-R-E-A-T Attenuator—Protection for Motorists and Automatic Gate

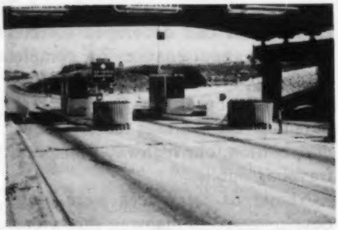

FIG. 29.—HI-DRO Cell Clusters—Protection at Toll Booths

increase in most line items for 1978: (1) Deaths—51,900; (2) disabling injuries—2,000,000; (3) costs—$35,800,000,000; (4) motor-vehicle mileage—1,525,000,-000,000 (5) death rate per 100,000,000 vehicle miles—3.40; (6) registered vehicles in the United States—159,400,000; and (7) licensed drivers in the United States—143,100,000.

Applications

In the past 15 yr, mathematical simulation techniques and computers have made possible solutions to the multifacet, dynamic problem associated with vehicle collisions. Heretofore only simplistic treatment of the problem was possible.

The application of engineering principles has made possible a wide variety of solutions which consider a wide range of vehicle weights, impact angles and speeds, deceleration forces, and dynamic material properties. As a result, highway appurtenances have been designed and installed which either "fail-safe"

or safely redirect errant vehicles which impact them. These include: (1) Breakaway luminaire and sign supports; (2) guardrails; (3) median barriers; (4) bridge rails; and (5) impact attenuators.

The vehicle crash management technology developed to date has encouraged research endeavors. Among such efforts are studies for "fail-safe" utility poles, self-restoring barriers for passenger vehicles, and forgiving highway appurtenances for school buses, intercity buses, and heavier transport trucks.

Conclusions

This paper dealt primarily with those safety aspects of roadside appurtenances which have been developed during the past 15 yr and some currently under development and test. They are basically "fail-safe" or redirective systems designed to reduce fatalities, personal injury, and property damage in low- and high-speed impacts by either small- or large-size passenger cars (28,29).

Design criteria for acceptable levels of safety were presented. Examples of roadside appurtenances were presented which highlighted the use of a large variety of materials such as steel, aluminum, conventional and light weight concrete, plastics, water, and sand. The technology transfer for achieving greater safety from one developed structure to other applications was shown. Uses of safety appurtenances on completed highway projects as well as temporary safety appurtenances in construction zones were considered.

Cooperation among Federal, state, and the private sector establishments in pursuing the various safety developments was identified. With continued cooperation, our highway transportation system will become even safer for the traveling public.

Numerous references were given for those interested in further pursuing developments in highway safety construction.

Acknowledgments

An acknowledgement with sincere appreciation is made of the many individuals in government, in industry, and in research and technical publication organizations who contributed so much in making our highway transportation system one of the world's safest.

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Simulation Model Validation: Airport Applications

By William J. Dunlay, Jr.,[1] M. ASCE

(Reviewed by the Air Transport Division)

Introduction

Simulation models have been used extensively to aid the planning and design of transportation facilities. Airport operations in particular have been the subject of many simulation attempts.

In most cases, the simulation model developer claims that his model has been "validated," but there is much confusion about this term. Some validations are detailed graphical or tabular comparisons of model outputs and corresponding observed, real-world data; others are only simple statements that the model estimates are reasonable and satisfactory.

The intention of this paper is to reduce some of the confusion about what is involved in the validation of a simulation model and to suggest guidelines for the documentation and disclosure of a simulation model to model developers and potential model users. The concepts described apply to simulation models of any mode of transportation. As will be explained, however, there are certain peculiarities of airport simulations that must be considered.

Purpose.—The purpose of this paper is to present basic principles for validating a fast-time stochastic or Monte Carlo simulation model of airport operations. The emphasis is on organizational procedures and statistical considerations in validation.

Scope.—The concepts and the methodology described in this paper apply to airport simulation models in general, e.g., landside models, airfield models, airspace models. To focus the analysis, however, it is assumed (without the loss of generality) that the simulation model involves, at least in part, aircraft operations.

Fundamentals of Validation.—Validation is especially important in computer simulation models. Unlike analytical models, which can usually be fully-described by a set of equations or by a short modifier (e.g., stochastic, deterministic,

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queueing, linear programming), simulation models defy easy characterization. In fact, the term *simulation model* conveys virtually no information about the inner workings of a model. True, there are certain accepted categories of simulation models (e.g., discrete event, discrete time, sequential calculation), but they add very little to one's insight into a model.

The validation of any computer simulation model of a complex system is very difficult. Naylor, et al., state that "the problem of verifying simulation models remains today perhaps the most elusive of all the unresolved problems associated with computer simulation techniques" (7).

There are no universally-accepted criteria against which simulation models can be measured. Acceptance of a simulation model must be based on the types of decisions that the application of the model is intended to support. Along these lines, Van Horn mentions two important characteristics of validation (8): (1) The objective is to validate a specific set of insights, not necessarily the mechanism that generated the insights; and (2) there is no such thing as "the" appropriate validation procedure. Validation is problem dependent. Van Horn's point is that it is the major attributes of the particular processes to be simulated that must guide a validation.

Validation of a computer simulation model should include: (1) A check of the validity of the assumptions and logic of the model and; (2) a comparison of model estimates with real-world observations. There is very little disagreement among modelers that both of these steps are required. De Neufville states that

> statistical analysis cannot be a sufficient test of any model. The validity of a systems model also rests on the plausibility of its a priori theoretical base (1).

It is not sufficient to test just the goodness-of-fit of the model to observe data. Naylor states that

> the ultimate test of a computer simulation model is the degree of accuracy with which the model predicts the behavior of the actual system (which is being simulated) in the future (7).

Unfortunately, because one cannot observe the future, it is impossible to directly validate the predictive capabilities of a simulation model. Instead, one must rely on the evidence of how well the model fits observable data, and how well the logic and assumptions of the model can be extended to situations that cannot be observed.

Validation is difficult even in situations in which real-world data are easy to observe. As Naylor has stated, "The problem of model validation becomes even more difficult if the available data about the 'actual' behavior of the world is itself subject to error" (7). This is certainly true of simulation models of airport operations, in which observed values of delays, travel times, etc., are subject to significant field measurement errors.

Key Principles and Steps in Validation.—To promote a better understanding of simulation model validation, the following key principles of validation are suggested:

1. A variety of comparisons—not just one—should be made between model estimates and real-world measures. In this way, one can weigh the whole set of evidence in deciding whether or not the model can satisfactorily approximate measured data.

2. The decision about a model's acceptability for its intended application is a subjective one based on a combination of statistical hypothesis testing and just "eyeballing" certain aspects of the model's outputs, logic, and sensitivities.

3. A committee should be established to oversee the validation. Van Horn suggests that such a committee is part of an ideal validation (8):

> Ideally a comparison test should handle nonstationarity, compensate for noisy data, simultaneously evaluate a number of output measures and work for small samples. Does such a test exist? The answer is yes if one is willing to define test very broadly. The test is simple. Find people who are directly involved with the actual process. Ask them to compare actual with simulation output.

4. Hypothesis tests should be used in such a way that they do not force a decision on acceptance or rejection on the basis of an arbitrary significance level but, instead, simply supply the decision maker with quantitative measures of goodness-of-fit of the model's estimates.

Three steps to be followed in the validation of an airport simulation model are proposed: (1) Evaluation of the model's logic and assumptions, the scope and kinds of inputs, and the scope and kinds of outputs; (2) sensitivity analysis of model estimates to changes in certain key inputs and assumptions; and (3) comparison of model estimates with the best available real-world measurements. These steps are described in the sections that follow.

Logic and Assumptions, Inputs, and Outputs

Model Logic and Assumptions.—The model logic consists of a set of relationships among the variables of the airport system, as implied by the way the model manipulates these variables. These relationships and manipulations should be evaluated against one's knowledge and understanding of airport operations. To facilitate such an evaluation, model logic flow charts should be provided that "walk the reader through" the simulation.

It is often desirable to check one of the very basic elements of a simulation model's logic (namely, its arithmetic) by relaxing all of the stochastic assumptions of the model and using it to solve very simple (even trivial) hypothetical examples that can be checked by hand or by using simple deterministic models. This is called "model verification."

Two kinds of assumptions are made in any simulation model: (1) Simplifying assumptions; and (2) statistical assumptions. As abstracts of the real world, simulation models necessarily involve simplifying assumptions. These assumptions should be clearly enumerated.

There are three types of statistical assumptions. The first is whether a given quantity is assumed to be a fixed constant or a random variable. The second

is the probability distribution of each random variable. The third concerns the statistical dependencies or correlations among the various random variables.

Scope and Kinds of Inputs.—At least five questions should be asked in the evaluation of inputs required for the application of an airport model:

1. Are the inputs sufficient to represent the operations at an airport?
2. Are the inputs sufficient to reflect the important parameters of the objectives of the simulation model application?
3. Are the inputs sufficiently sensitive to local (airport-specific) conditions?
4. How difficult is it to obtain the required inputs? Is this difficulty excessive considering the expected benefits of applying the model?
5. How sensitive is the required set of inputs to possible future changes in technology?

Scope and Kinds of Outputs.—The quality of the outputs can, of course, be no better than that of the inputs and model logic. Nevertheless, one evaluation criterion for a model is the scope and format of its output. Does the model, e.g., provide delay information by cause, location, time of day, type of aircraft, airline, etc.? Is there sufficient flexibility in cross-tabulating and aggregating the outputs for subsequent analysis? In short, are the model outputs pertinent to the kinds of decisions the model is intended to support?

Sensitivity Analysis of Model

This second step of model validation is aimed at exploring certain inner properties of a simulation model. A sensitivity analysis usually involves evaluating the change in one or more key output variables resulting from systematic changes in one or more inputs.

There are several reasons for performing a sensitivity analysis. One is to determine whether the model outputs are sensitive to small changes in particular input; if so that input will have to be measured very accurately, and assumptions about it will have to be closely scrutinized.

A second reason for a sensitivity analysis is to evaluate how extrapolatable the model is to new, nonobservable situations by systematically varying one or more inputs and then judging the resulting output changes predicted by the model against what one would expect to happen. Thus, the sensitivity of a simulation model is a very important aspect of its logic. One may wish, e.g., to examine the delays that result from incrementally adding new aircraft to the existing demand. Or, one may wish to determine whether the model can reasonably predict the effect of a major change (e.g., a sudden drop in ceiling and visibility) that is known from past experience at a particular airport to have a dramatic effect on delays.

A third reason for a sensitivity analysis is to evaluate how sensitive the model estimates are to simplifying and statistical assumptions. Suppose, e.g., that an assumption is made that cannot be verified. If it turns out that the model output is very sensitive to small deviations from that assumption, then an effort should be made to check it further (and possibly revise it).

Empirical Validation of Model Outputs

Validation Variables.—As stated earlier, a variety of comparisons should be

made between model estimates and actual observed data; in particular, comparisons based on the following variables are recommended: (1) Delays to arrivals and departures; (2) travel times of arrivals and departures; (3) aircraft flow rates; and (4) arrival and departure queue sizes.

Delays are the most important and also the most complex variables. It is difficult to separate: (1) Delays resulting from congestion at the destination airport; and (2) delays resulting from congestion en route or other causes not related to the airport.

A second and closely-related problem is that those delays attributable to the airport are not all incurred at a single point. Such delays, e.g., might be incurred en route at the advice (e.g., speed control or path stretching) of an air traffic controller or airline dispatcher. That is, delays can back up to various distances before an aircraft arrives at the terminal airspace.

Data on delays to aircraft are dependent on the specific runway configurations and weather conditions in effect when the delays occur. There are many problems

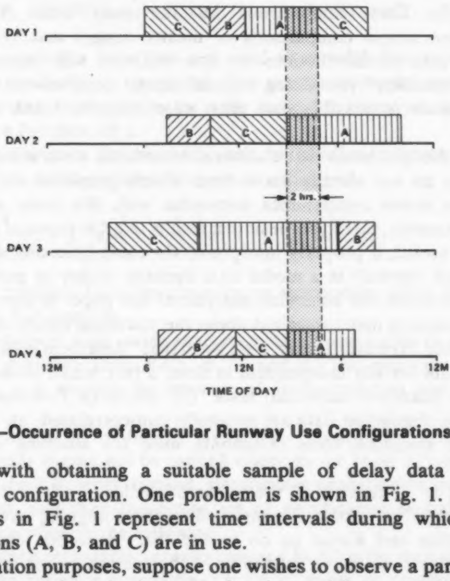

FIG. 1.—Occurrence of Particular Runway Use Configurations A, B, and C

associated with obtaining a suitable sample of delay data for a particular runway-use configuration. One problem is shown in Fig. 1. Segments of the shaded bars in Fig. 1 represent time intervals during which three runway configurations (A, B, and C) are in use.

For validation purposes, suppose one wishes to observe a particular configuration during approximately the same time interval for a sample of *n* days. Note that on all four days shown in Fig. 1 there is a common 2-h period (indicated by dashed lines) during which Configuration A is used. Configuration B is not so repetitive; it is used, however, during a common 2-1/2-h period on Days 2 and 4. Configuration C is even less repetitive than Configuration B.

Another complication is that the delays encountered in one time interval (especially at the beginning of the interval) with a given runway configuration are dependent on the runway configuration in use in the immediately preceding time interval. In Fig. 1, e.g., Configuration A is preceded by Configuration

B on Day 1, and by Configuration C on the other three days. Aircraft may be left over from a lower-capacity runway configuration in the preceding interval, which would mean that delays at the beginning of the interval would be higher than expected and would gradually decline. On the other hand, if the runway configuration in the preceding interval had a greater capacity, it would take a while for the delays to build up to be representative of the current configuration.

Because of these problems, it may be difficult to obtain adequate samples of delay data for time intervals on successive days that are assumed to constitute independent and identical conditions. An alternative would be to have each sample consist of a time history of delays measured on one day, i.e., delays in successive time intervals on the same day. In this case, different days would represent distinct samples to be treated separately rather than averaged together. Such a treatment is termed a time-series analysis. Important statistical considerations related to time-series analysis are examined in the following.

Statistical Considerations.—One of the most complex and misunderstood aspects of simulation model validation is the statistical comparison of model estimates with observed data. These comparisons can take many forms. According to Van Horn, "Often simple comparisons of means, ranges and variances and graphical comparison of distributions or time behavior will capture most of the available information" (8). Going beyond simple comparisons and testing statistical hypotheses is possible, but great care must be taken in selecting appropriate tests.

One school of thought holds that statistical hypothesis tests will not tell you anything that you do not already know from simple graphical and "eyeball" comparisons. The writer sympathizes somewhat with this point of view but with one proviso, namely, that hypothesis test results (e.g., in terms of significance probabilities) are useful, if properly interpreted, in communicating the analyst's confidence (or lack thereof) in a model to a decision maker or potential user. It is toward that end that the statistical analysis of this paper is directed.

The literature contains many warnings about the statistical nature of the output of simulation models. Hsu and Hunter, e.g., warn that "data from many simulation models are often not serially independent in time, a fact which seriously affects the validity of the [standard statistical] tests" (5). Similarly, Fishman and Kiviat point out that "As simulation data are generally autocorrelated, an investigator cannot apply the statistical tools commonly used for studying independent observations" (3).

Autocorrelation is a measure of the linear dependence of a process on its past. The autocorrelation problem mentioned in the foregoing caveats cannot be ignored. Fishman and Kiviat go on to say that "Ignoring autocorrelation is clearly unacceptable, since the reliability of the sample means and variances are thereby overestimated" (3). Besides, as Hsu and Hunter point out, "serial correlation in time is itself an important characteristic . . ." that can be compared statistically with the corresponding serial correlation structure of the real-world data as part of the validation process (5).

Estimating Autocorrelation.—Before applying detailed statistical hypothesis tests, one can gain insight into a time series by estimating its autocorrelation function. Consider a time series, Y_t, that has the properties of a covariance-stationary process, namely, neither the covariance structure nor the expected value of the time series is a function of time. The autocovariance function of Y_t

is defined as

$$\mathrm{Cov}\,[Y_t, Y_{t+s}] = E\{[Y_t - E(Y_t)]\,[Y_{t+s} - E(Y_{t+s})]\} \quad (1)$$

The assumption of stationarity implies that Cov (Y_t, Y_{t+s}) depends only on s and not on t. The autocovariance function can be estimated using the following estimator for the "sample covariance of lag s"

$$c_s = \frac{1}{n-s} \sum_{t=1}^{n-s} (Y_t - \bar{Y})(Y_{t+s} - \bar{Y}) \quad (2)$$

Note that c_0 (i.e., c_s for $s = 0$) is the sample variance of Y_t. The autocorrelation function, ρ_s, obtained from

$$\rho_s = \frac{c_s}{c_0} \quad (3)$$

is a measure of the linear dependency of Y_t on its past history. The function ρ_s is equal to unity for $s = 0$ and lies in the interval $(-1, 1)$ for all other values of s.

The autocorrelation functions are used in a validation to gain insight into the two time series being compared. One can also compare the autocorrelation functions for the model estimates with those for observed data after plotting each one as a function of s.

The autocovariance function, c_s, can be used to estimate the variance of the mean of an autocorrelated time series as follows

$$\mathrm{Var}\,[\bar{Y}] = \frac{1}{n} c_0 + 2 \sum_{s=1}^{k} \left(1 - \frac{s}{k}\right) c_s \quad (4)$$

in which the problem becomes choosing an appropriate value of k. This problem is treated by Fishman (3).

Further details of time series analysis methods applicable to airport simulation models are available from Refs. 4, 5, and 6. The purpose of the foregoing examination is to explain the concept of autocorrelation and how to compute it.

Decisions Based on Statistical Hypothesis Tests.—The results of statistical hypothesis tests should not be judged solely on the basis of a priori significance levels, e.g., the 0.05 level. Instead, significance probabilities should be estimated for each test, and then the entire set of significance probabilities should be judged as described in the following.

A significance probability (usually denoted as P_I) is the probability of obtaining a value of an inferential statistic, i.e., a χ^2 value or t value, as large as the one computed in the test, given that the hypothesis tested is actually true. (Recall that the general nature of the hypotheses tested in a validation is that there is no difference between the model estimates and the observed data.) If this probability is very small, then one would tend to reject or at least to suspect the hypothesis. If the significance probability is not small, then one would tend not to reject the hypothesis. The conclusion in the latter case might be that the differences obtained in the test are due to chance rather than to defects in the model. No precise universal definition of "small" can be offered.

This definition is a matter of judgment and confidence in the statistical methods employed.

Further insight into the test results may be obtained by considering the results of a whole set of tests. Suppose, e.g., that test results in the form of significance probabilities P_{ij}, in which i = the test number, and j = the comparison variable, are tabulated as in Figure 2. If the hypotheses tested were all really true, then at any arbitrary significance level, α, one would expect the number of tests that failed, i.e., the number of P_{ij}, $<\alpha$, to not exceed $(100\alpha)\%$ of the total number of tests kl. Thus, not more than about 5% of the P_{ij} should be less than 0.05, not more than about 10% should be less than 0.10, etc., by definition of significance probability. If k is very large, these percentages are best checked within each column: $P_{ij}, \ldots, P_{kj}; j = 1, \ldots, l$. These are only very rough guidelines, because the tests are not all the same, and the total number of tests is likely to be small.

The hypothesis test results are only one measure of the model's ability to simulate airport operations. The results of these statistical comparisons must

		Comparison Variable				
		1	2	j		ℓ
	1	P_{11}	. . .	.	. . .	$P_{1\ell}$
	2	.		.		
	.	.		.		
	.	.		.		
Test No.	.	.		.		
	i		. . .	P_{ij}		
	.	.		.		
	.	.		.		
	.	.		.		
	k	P_{k1}	. . .	.	. . .	$P_{k\ell}$

FIG. 2.—Significance Probabilities for Set of Hypothesis Tests

be weighed along with other evidence (e.g., graphical and tabular comparisons, results of the sensitivity analyses, and the evaluation of model logic) in making the final judgment about the adequacy of the model for its intended applications.

Model Replications and Convergence.—The internal convergence of stochastic (Monte Carlo) simulation model estimates is an issue separate from (but related to) the validation comparisons with observed data. A simulation model, e.g., can produce average values with extremely high precision, as measured by the standard error of the mean, but this says nothing about the accuracy of those average values in an absolute sense (i.e., relative to corresponding observed values).

The question of concern here is, "How many independent replications of the model (each with a different random number seed) are required to obtain a desired degree of precision in model estimates of mean values of response

variables?" The degree of precision will be assumed to be expressed as a confidence interval (e.g., a 95% confidence interval).

Suppose that the model is run n times, each with the same input data but with a different random number seed. Also suppose that from each run an estimate is obtained of an output variable (e.g., average delay or aircraft flow rate: $L_1, L_2, \ldots, L_n$). The point estimate of the expected value of the n replications is

$$\hat{L} = \frac{1}{n}\sum_{i=1}^{n} L_i \qquad (5)$$

and the variance estimator is

$$s^2 = \frac{1}{n-1}\sum_{i=1}^{n} (L_i - \hat{L})^2 \qquad (6)$$

If n is sufficiently large, e.g., at least twelve, the sample average $\hat{L}$ may be assumed to have a normal distribution with variance

$$s_L^2 = \frac{1}{n}\,\frac{\sum_{i=1}^{n}(L_i - \hat{L})^2}{n-1} = \frac{1}{n}s^2 \qquad (7)$$

The assumption of normality is supported by the central limit theorem even if the L_i are not normally distributed. Also, the population variance is not usually known a priori but must be approximated by the sample variance.

The foregoing assumptions allow us to obtain a confidence interval estimate for $\hat{L}$ as

$$\hat{L} \pm Z_\alpha s_L \qquad (8)$$

if n is 30 or larger, or

$$\hat{L} \pm t_\alpha(n-1)s_L \qquad (9)$$

for n less than 30; in which Z_α is from the standard normal tables; $1 - \alpha$ = the confidence level; and $t_\alpha(n-1)$ is from a table of Student's t distribution with $n - 1$ degrees of freedom. Use of the t statistic implies the additional assumption that the L_i are normally distributed. The test is, however, fairly robust to departures from normality.

The 95% confidence interval, $\hat{L} \pm Z_{0.05}s_L$ or $\hat{L} \pm t_{0.05}(n-1)s_L$, can be specified in advance as either an absolute value, say $\hat{L} \pm A$ minutes of delay, or as a fraction B of the mean value (e.g., $\hat{L} \pm B\hat{L}$). In this latter case, which is a popular way to specify desired convergence, it must be realized that $\hat{L} + B\hat{L}$ is a random variable and not a fixed range. Thus, the usual equations for such intervals (e.g., Eqs. 11 and 13) are only approximate because they ignore this fact.

In the cases in which $n > 30$, the Z_α value is usually assumed to be based on a known, fixed population variance $\sigma^2_{L_i}$, even though we estimate it with s. Therefore, the required number of replications to achieve a specified precision, α, is given by

$$n \geq \left(\frac{Z_\alpha s}{A}\right)^2 \quad \ldots \quad (10)$$

$$\text{or} \quad n \geq \left(\frac{Z_\alpha s}{B\hat{L}}\right)^2 \quad \ldots \quad (11)$$

Similarly, even when $n < 30$, it is usually assumed that the estimator, s, is not a function of n. Therefore, the required number of replications can be obtained from

$$n \geq \left(\frac{t_\alpha(n-1)\,s}{A}\right)^2 \quad \ldots \quad (12)$$

$$\text{or} \quad n \geq \left(\frac{t_\alpha(n-1)s}{B\hat{L}}\right)^2 \quad \ldots \quad (13)$$

Note, however, that in Eqs. 12 and 13, $t_\alpha(n-1)$ is itself a function of n. Therefore, n must be estimated by trial (i.e., assume a value, say n^*, plug in $t_\alpha(n^* - 1)$, solve for n, and check to see if n^* is sufficiently close to the n computed; if not, repeat until n^* and n are sufficiently close).

How rapidly the model converges for any given n depends on how many aircraft are processed in each replication which, in turn, affects the total number of variates randomly drawn (assuming a fixed number of variates per aircraft) in each replication. Thus, it is not possible, and in fact might be wasteful, to make any blanket statements about how many replications are necessary to achieve the desired convergence. Instead, a relationship should be developed between the number of replications, the number of aircraft processed per replication, and the desired confidence interval. Such a relation could be depicted graphically as in Fig. 3 (schematic only) for a given response variable expressed in units of minutes.

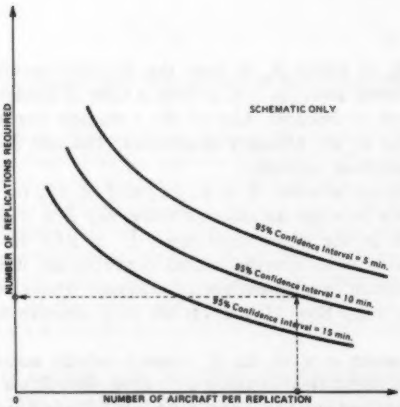

FIG. 3.—Model Convergence as Function of Run Size for Given Comparison Variable

Note that the curves of Fig. 3 are essentially contours of equal-confidence interval values. These can be plotted as follows:

1. Run the model for a large number of replications, say at least 30, for each activity level and for the response variable with the largest variance. Compute cumulative averages, Eq. 1, for each response variable at every five replications. Also, compute the cumulative sample variance, Eq. 2, at every five replications. This will yield the 95% confidence interval for a given number of replications for each activity level.

2. Create a grid made up of horizontal lines at five-replication intervals and vertical lines corresponding to the different activity levels.

3. Compute confidence intervals for each combination of the number of replications and the number of aircraft processed, and record these at the corresponding intersections on the grid.

4. Interpolate contours on the grid for convenient, rounded-integer-valued confidence intervals (e.g., see the 5-min, 10-min, and 15-min contours of Fig. 3).

This may seem a tedious process, but the result is valuable, namely, a guide for choosing the number of runs for using the model, given specifications on convergence, the approximate activity level under consideration, in aircraft per hour, and the length of time period simulated, in hours. Note that the abscissa of Fig. 3 is the product of these last two quantities.

Conclusions

The main conclusions of this paper are:

1. The internal workings and assumptions of a simulation model should be disclosed in a validation; statistical comparisons of outputs with observed data are necessary but not sufficient.

2. Validation of a computer simulation model is (and should be) largely a subjective process based on a variety of qualitative and quantitative evidence on the model's logic, goodness-of-fit, and predictive power.

3. Autocorrelation of simulation model outputs and corresponding observed data on time histories of airport operations should be considered in statistical hypothesis tests of the two series.

4. An analysis of model convergence should be performed to obtain a guide on the number of replications of the model required for a desired level of confidence.

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Appendix II.—Notation

The following symbols are used in this paper:

A = confidence interval, in minutes of delay;
B = confidence interval, in fraction of mean delay;
c_s = sample covariance;
$E\{x\}$ = expected value of x;
k = variance calibration variable;
L = arbitrary output variable;
$\hat{L}$ = expected value estimator;
n = number of replications;
P_t = significance probability;
s = time lag or standard deviation estimator;
t = time or Student's t distribution;
Y_t = time series function;
Z = variate of standard-normal distribution;
$\mathrm{Cov}[x,y]$ = covariance of x and y;
$\mathrm{Var}[y]$ = variance of y;
α = significance level; and
ρ_s = autocorrelation function.

PROSPECTING FOR WIND: WINDMILLS AND WIND CHARACTERISTICS[a]

By Robert N. Meroney,[1] M. ASCE

(Reviewed by the Aerospace Division)

INTRODUCTION

The windmill, that familiar if not always reliable workhorse of mankind, seems headed back into the whirl again. Visions of wind-driven power sources have tantalized mankind for centuries; however, only recently has anyone considered seriously producing by such means a significant fraction of mankind's electrical power (say 6.6 quads of energy by the year 2020). To reach this optimistic goal requires careful consideration of the characteristics and availability of the energy source itself, i.e., the wind. Effective utilization of wind energy will require estimates of wind characteristics related to: (1) Wind turbine aerodynamics, hardware, and tower design; (2) energy, climatology, and resource estimates over large mesoscale size regions; (3) dependable and cost-effective methodologies for pre-evaluating specific sites; and (4) forecasting wind conditions for large turbine system operations. A review of the history of wind characteristics research prior to 1970 has been prepared by the writer (13). Since 1976 the Wind Characteristics Program Element of the Federal Wind Energy Program has been coordinated by the Pacific Northwest Laboratory (PNL) at Richland, Washington (18).

Past experience with power generation by large windmills suggests that among the most important factors controlling success or failure of these systems is site selection and presiting evaluation—the subject of the second and third areas previously listed. Incorrect placement on a site sheltered by buildings, terrain, or agricultural growth may drop performance to one third of the original expectations. Conversely, the appropriate hill or ridge shape may amplify power available at a given height by an order of magnitude above that over flat terrain! Recent insights on the influence of terrain suggest that there is significant wind energy available in coastal regions and over complex mountain topography.

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Site selection procedures in such instances include statistical climatology, numerical simulation, and physical simulation in meteorological wind tunnels. A review of "wind prospector" methodologies is the subject of this paper.

The second section considers historical siting insights which persist from previous Wind Energy Conversion System (WECS) experience. A third section is concerned with the analysis of wind energy potential over mesoscale size regions (1,000 km × 1,000 km). The development of accurate presiting methodologies is considered in the concluding discussion.

Historical Perspectives

Some 500,000 small wind-electric systems are known to have been built in the United States, most prior to 1945. These electric generating systems were sited on the basis of rough "rules of thumb," generally a minimum height above obstacles within a specified radius. Installations were encouraged where there was an "intuitively understood 'minimal availability' of wind energy" (1).

World-wide experience prior to 1970 with some ten large wind turbines exceeding nominal capacities of 100 kW resulted in a qualitative consensus on wind-site evaluation over low-to medium-height ridges or hills (11).

1. Ridges should be athwart the principal wind direction, but high velocities are not likely on upwind foothills.
2. Hill tops should not be too flat, slopes should extend all the way to the summit.
3. A hill on the coast as opposed to an inland hill surrounded by other hills is more likely to provide high winds, i.e., unobstructed upwind.
4. Speed up is greater over a ridge of given slope than over a conical hill of the same slope.
5. Speed up over a steep hill decreases rapidly with height.
6. The optimum hill slope is probably between 1:4 and 1:3 with 1:3.5 best (h/L between 0.5 and 0.67).
7. Topographical features in the vicinity of the hill produce the structure of the flow over it.
8. Frenkiel (Ref. 5) ranks sites based on the uniformity of the summit and profile (see Table 1).

TABLE 1.—Rank of Sites Based on Uniformity of Summit and Profile

Quality (1)	$R = {}^{u}40\ m/u_{10m}$ (2)	α (3)	Slope (4)	h/L (5)
Optimum	$R < 1.05$	0.0	1:3.5	0.57
Very good	$1.05 < R < 1.10$	0.07	1:6 smooth regular	0.35
Good	$1.1 < R < 1.15$	0.1	1:10	0.20
Fair	$1.15 < R < 1.21$	0.14	1:20 smooth 1:6 regular	0.10
Avoid	$1.21 < R$	>0.14	>1.20 <1:2	<0.05 >1.0

9. Hills with slopes greater than 1:3 should probably be avoided.
10. Vertical wind speed above a summit does not increase as much with height above ground as over level terrain.

A great deal has been learned about atmospheric flows since the early siting exercises. Research sponsored by the Federal Wind Energy Program has resulted in the preparation of several handbook volumes which attempt to deal systematically with the separate or combined influence of terrain features, surface roughness, buildings, etc. (6,14,17).

National and Regional Wind-Energy Potential

A study comparing, evaluating, and synthesizing three previous independent national assessments of wind-energy potential was reported by Elliot (3). This

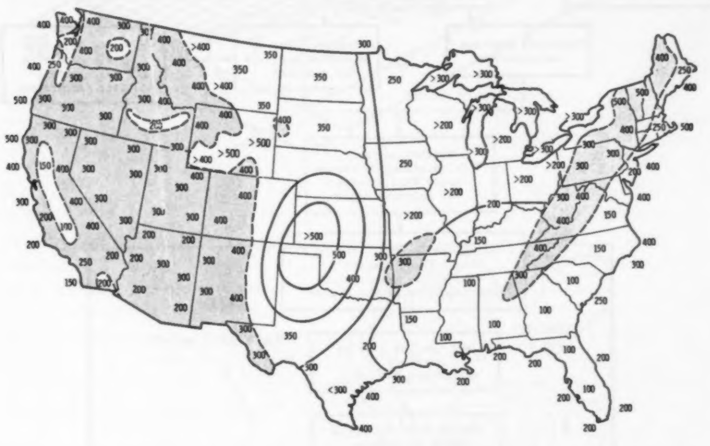


FIG. 1.—Mean Annual Wind Power (*W* per square meter) Estimated at 50 m above Exposed Areas [over Mountainous Regions (Shaded Areas) Estimates are Lower Limits Expected for Exposed Mountain Tops and Ridges (3)

report examined some of the inherent problems with respect to representativeness and reliability of the surface and rawinsondes wind data, techniques employed in the vertical extrapolation of wind power, in the estimation of wind power over mountainous and offshore areas and areas of sparse data, and in the analysis and interpolation of the values. The refined result for mean annual wind power at a typical 50-m hub height above exposed ground over the contiguous United States is shown in Fig. 1. The estimates are considered to be lower limits for exposed sites; however, a few areas may have 50%–100% greater wind power. Since in mountainous areas the estimates are based on the climatology of the winds aloft, some isolated ridges or gaps may provide power a factor of two or three greater. An area where annual mean wind power (w/m^2) exceeds 400 is considered very good.

To meet the need for more detailed analyses over regions below areas of 100 km × 100 km, a series of data-screening techniques have been proposed. Analytical techniques, which involve the use of aerial photography and satellite imagery of eolian geomorphological features, suggest that stabilized eolian features in arid climate areas are reliable indicators of wind patterns (10). About 52% of the 17 western states are susceptible to eolian action.

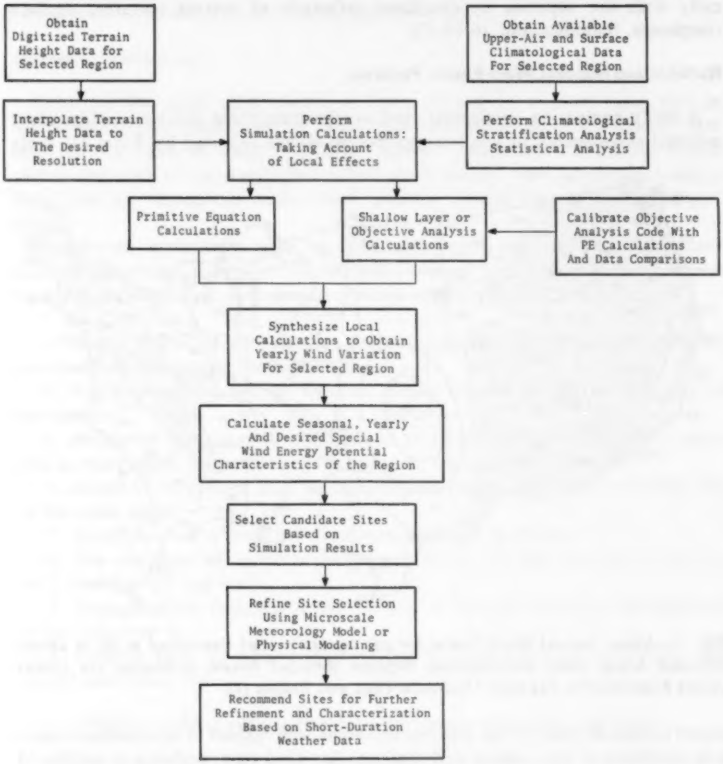


FIG. 2.—WECS Siting Methodology

Joint climatological-numerical screening methods appear suitable for mesoscale area evaluation. These methods employ mathematical models of meso and micrometeorology over coastal or complex terrain to extend clomatological data from meteorological stations where records are available and predict the climatology of sites, within the region of interest, where data are unavailable. A schematic of such a siting methodology is shown in Figure 2.

Three classes of numerical models have been considered for such a methodol-

ogy: (1) Full primitive equation (PE) models; (2) objective analysis (OA) models; and (3) shallow fluid (SF) or Lavoie models (4,7,8,15,16). Typical primitive equation models solve the conservation equations for momentum, energy, and moisture transport. The equations usually account for advection, Coriolis forces,

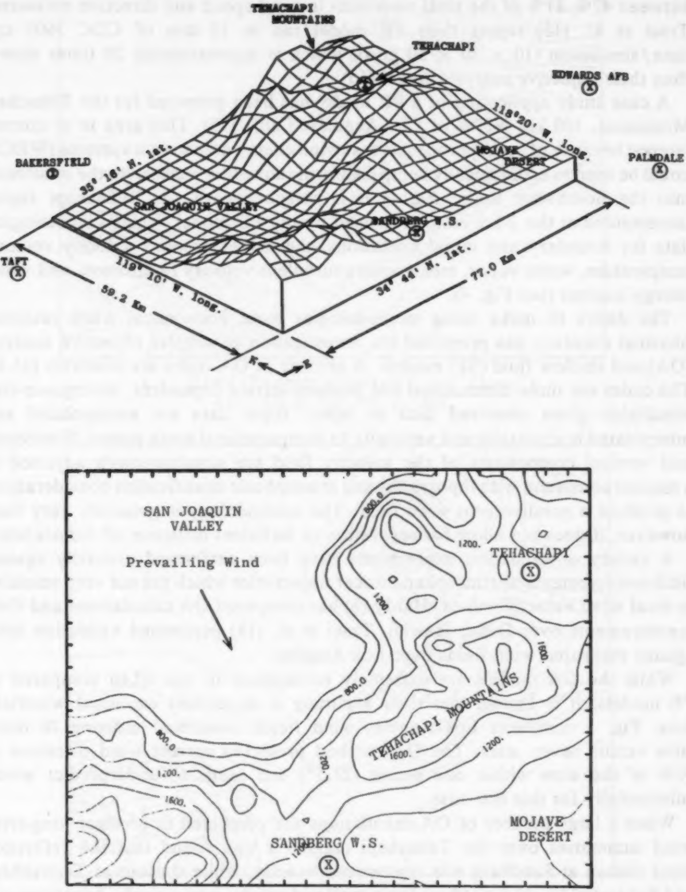


FIG. 3.—Terrain Perspective of Tehachapi Region Traci et al. (15)

turbulent heat, momentum and moisture transport, and radiation. The most sophisticated versions utilize turbulent models which account for density stratification, turbulence nonequilibrium, and history effects. Typically they use a terrain following grid system with expanded grid resolution near the surface.

Various approximations do not usually permit prediction of separation or reduction of resolution to turbine-size scales.

Validation of PE methods has been difficult. Vukovich (16) found that data-type inconsistencies inherently present in the field experiments chosen for validation forestalled positive conclusions of simulation (16). The model tested explained between 47%–81% of the total variations in wind speed and direction measured. Traci et al. (15) report their PE model ran in 10 min of CDC 7600 cpu time/simulation (10 × 30 × 30 grid), which is approximately 20 times slower than their objective analysis model (15).

A case study application of a PE model has been prepared for the Tehachapi Mountains, 100 km NNW of Los Angeles, Calif. (15). This area is of current interest because of recent proposals that wind energy conversion systems (WECS) could be used to economically pump California aqueduct water over the mountains into the Southwest basin. Fig. 3 is a perspective of the Tehachapi region incremented at the 3-km resolution used by the PE model. Using climatological data for boundary and initial conditions, the model predicts velocity vectors, temperature, water vapor, mean-square turbulent-velocity fluctuation, and wind-energy content (see Fig. 4).

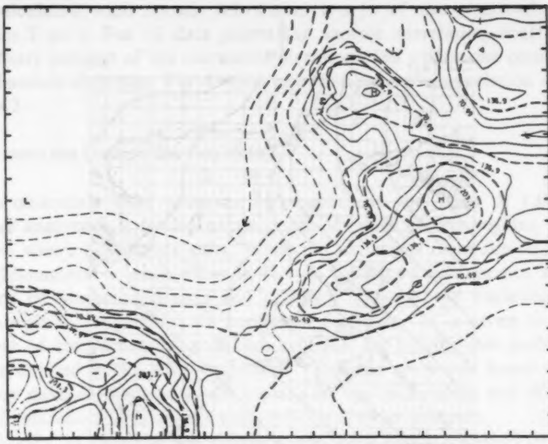
The desire to make siting methodologies more economical while retaining physical accuracy has prompted the investigation of simpler objective analysis (OA) and shallow fluid (SF) models. A number of OA codes are available (15,8). The codes are three dimensional and produce terrain dependent, divergence-free windfields given observed data as input. Input data are extrapolated and interpolated horizontally and vertically to computational mesh points. Horizontal and vertical components of the velocity field are simultaneously adjusted in a manner consistent with topography and atmospheric stratification considerations to produce a nondivergent wind field. The methodology is generally very fast; however, it does not allow for separation or turbulent diffusion of momentum.

A variety of validation experiments have been performed primarily against field measurements of atmospheric tracer trajectories which are not very sensitive to local wind velocity values. Hardy (8) has compared OA calculations and field measurements over Oahu, Hawaii. Traci et al. (15) performed validation tests against measured wind fields over Los Angeles.

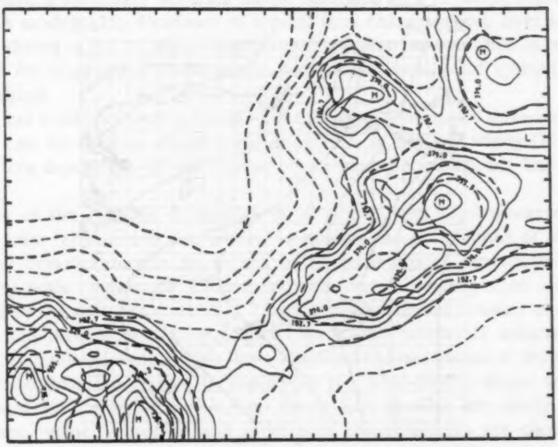
While the OA models are extremely economical to run when compared to PE models, it is known that their accuracy is dependent on initial windfield data. Fig. 5 compares Los Angeles wind fields calculated utilizing 26 input sites versus seven sites. The OA method predicted correct wind direction at 50% of the sites within one sector (22.5°) and tends to underpredict winds substantially for this test case.

When a large number of OA calculations are combined to produce long-term wind summaries over the Tehachapi region, it was found that the reference input station at Sandberg was reproduced exactly, while stations at Techachapi and Bakersfield were reproduced very well and poorly, respectively. In retrospect, this was not surprising since valley input data was limited.

The other class of simple meteorological models considered is the shallow fluid (SF) model. The active part of the atmosphere is represented by a single computational layer. Terrain shape, roughness, and thermal forcing can be treated at small expense. The method has an inherent lack of vertical resolution; however, combinations of OA and SF models look attractive (15).



a. Wind Energy Content (m^3/sec^3) at ~42 m to 63 m AGL



b. Wind Energy Content (m^3/sec^3) at ~131 m to 196 m AGL

FIG. 4.—Near Surface Wind Energy Content Contours: Tehachapi Flow Field Computation (1230 GMT, 11/15/74)

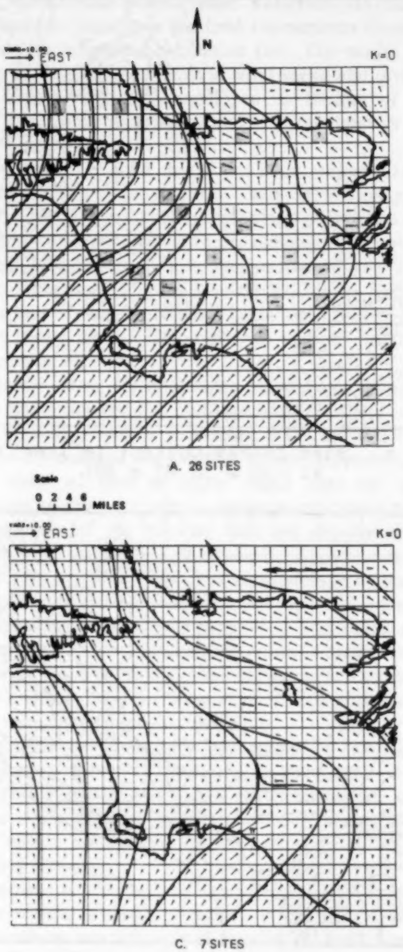


FIG. 5.—Objective Analysis Validation Test Using Los Angeles Data (1500 hours, 30 October 1974) Comparing Streamline Patterns as Function of Number of Meteorological Inputs (15)

Fosberg (4) compared an SF model to seven sets of field data. Six come from studies conducted in the Oregon Cascade mountains. Forty-five percent of the calculated wind speeds fell within 1 m/s of the observed, and 75% fell within 2 m/s. For 51 data points the sample correlation coefficient is $r = 0.60$. Sixty percent of the calculated directions fell within one compass point of the observed direction. For 43 data points the sample correlation coefficient is $r = 0.62$.

Site Screening and Localization Techniques

In the mesoscale wind program, large areas on the order of 1,000 km on a side are analyzed. It is also necessary to identify smaller regions within the large area where high winds exist. While the variations from site to site within several kilometers of each other in flat or rolling terrain are not significant, in regions of nonhomogeneous and complex terrain these variations may be large enough to jeopardize the economic feasibility of a given turbine site. A number of complimentary tools are available for solving this problem which might be applied in the following order: (1) Rules of thumb based on generic laboratory data; (2) biological indicators; (3) physical models; and (4) statistical methods with short-term on-site meteorological measurements.

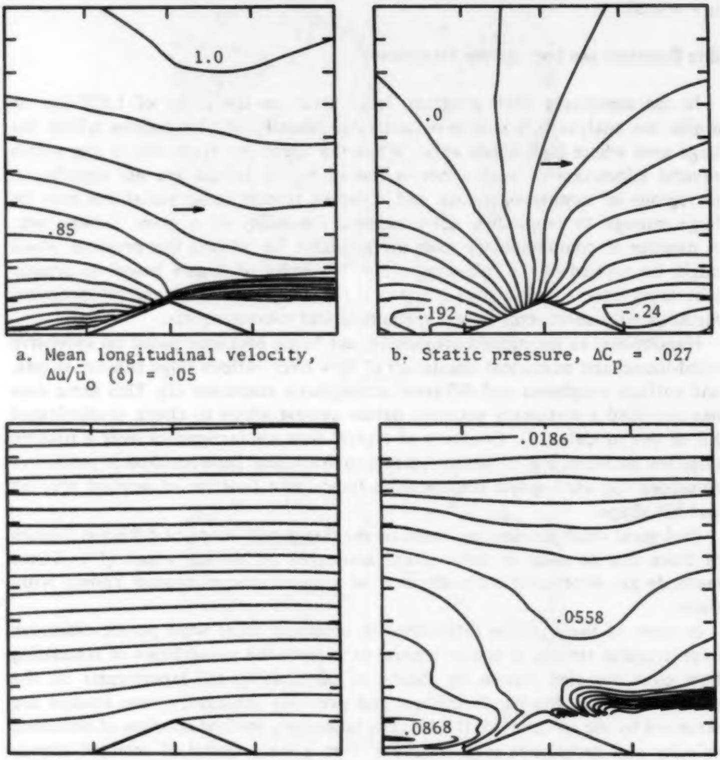
Handbooks, as mentioned previously, are being prepared based on extensive wind-tunnel and numerical simulation of flow over various ridge shapes, slopes, and surface roughness and different atmospheric stabilities (2). This same data has provided a stationary accurate datum against which to check sophisticated PE or OA models (15). Contours of typical flow characteristics over a triangle ridge are shown in Fig. 6. Measurements such as these have resulted in predictive equations for wind-speed amplification factor as a function of upwind profiles and hill shape.

Biological wind prospecting based on the flagging or shape of different species of trees can be used to detect local anomalies in surface winds (17). These methods are dependent on calibration of a given species against known wind fields.

In view of the extreme difficulties in obtaining local wind power estimates over irregular terrain, it is also natural to explore the possibilities of simulating flow over complex terrain by means of physical model experiments on the laboratory scale. Similitude criteria and previous laboratory case studies are reviewed by the writer et al. (11,12). The laboratory method consists of obtaining velocity and turbulence measurements over a scale model of selected terrain placed in a simulated atmospheric flow. The wind characteristics of the simulated atmospheric flow are chosen to reproduce the wind-profile shape and length of the equivalent prototype situation. Since field profiles are rarely available in advance, velocity profiles and turbulence characteristics are chosen to fit an equivalent class of conditions as recorded by earlier investigators over terrain of similar roughness.

The representative area studied by means of a laboratory model is located along the Rakaia River as it emerges from the Southern Alps, South Island, New Zealand. The primary terrain features consist of the Rakaia River Gorge which runs generally in a northwest-southeast direction. Gorge walls rise 180 m, surrounding hills rise to 460 m. To the south lies the Mount Hutt range

which climbs to 2,188 m. The range parallels the course of the Rakaia River in this area. To the north lies the Rugged Range but nearby Fighting Hill and Round Hill are the largest features. A model section 6,100 m wide by 18,300 m long centered over the Rakaia River Gorge was constructed to an undistorted scale of 1:5000.



a. Mean longitudinal velocity, $\Delta u/\bar{u}_o\ (\delta) = .05$

b. Static pressure, $\Delta C_p = .027$

c. Streamlines

d. Longitudinal turbulence intensity, $\Delta u'/\bar{u}_o(\delta) = .0062$

FIG. 6.—Contours of Flow Characteristics over Triangular Ridge, $h/L = 1/2$: Test Case 1 (2)

Climatological records obtained from stations somewhat removed from the area suggest moderate to very high wind energy suitable for wind energy conversion system sites. Local farmer and fishermen wisdom and folklore speak of incredible winds in the gorge canyon. Extended field measurement programs

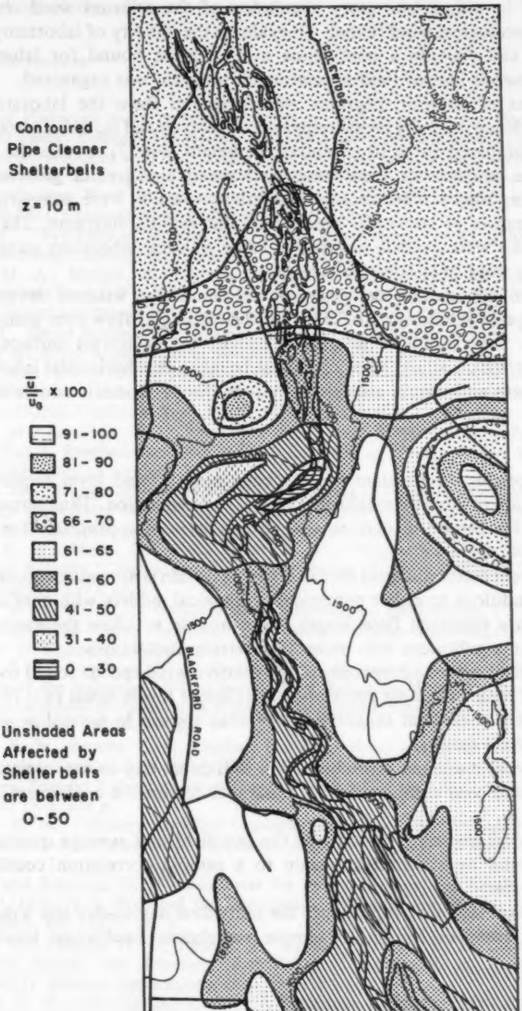


FIG. 7.—Horizontal Isotachs: Contoured Model, Rakaia Gorge, New Zealand (12)

are invariably expensive and time consuming, therefore a survey program was proposed to utilize laboratory simulation of the relevant wind characteristics in a meteorological wind tunnel. To evaluate the validity of laboratory simulation methods and provide a confidence measurement bound for laboratory data, a simultaneous limited field-measurement program was organized.

A series of contour diagrams were prepared from the laboratory velocity and turbulence intensity measurements into isotach and isoturb charts. Horizontal sections prepared for a 10 m equivalent height (Fig. 7) reveals the river valley and gorge consistently has lower wind speed and greater gustiness than the surrounding ridges. The laboratory simulation results were compared with field data by means of statistical correlation and scatter diagrams. The model and field results were used to assess the value of such laboratory experiments for predicting wind over complex terrain.

It would appear that the conventional simulation wisdom developed in the past few years is appropriate for physical modeling of flow over complex terrain. Since the flow region of interest is usually in the lowest surface layer ($z <$ 100 m) for WECS siting, great care must be taken that horizontal inhomogeneities in roughness and terrain are faithfully reproduced. Specific conclusions suggest that:

1. To produce equivalent wind speeds near ground level requires accurate reproduction of surface roughness, shape, and vegetation. Thus terraced models, adequate for certain dispersion simulations, are not appropriate for WECS site analysis.
2. Current meteorological data in complex terrain is not yet adequate to stipulate inflow conditions to either numerical or physical models with confidence. Thus an adequate approach flow length must provide to allow the surface layer to come to an equilibrium with underlying terrain undulations.
3. Physical modeling reproduced the relative wind speeds found over complex terrain by rank to sample correlation coefficient levels equal to 0.78–0.95 (these numbers are somewhat sensitive to the sites chosen to normalize and compare model to field results).
4. Physical modeling reproduced the individual day-to-day quantitative wind speeds found over complex terrain to sample correlation coefficient levels equal to 0.70–0.76.
5. Physical modeling reproduced the two-field day average quantitative wind speeds found over complex terrain to a sample correlation coefficient level equal to 0.81.
6. Physical modeling reproduced the individual day-to-day site wind directions found on complex terrain to sample correlation coefficient levels equal to 0.65–0.67.

Summary

Climatology statistics and simulation techniques are the "pick and shovel" of the modern mountain meteorologist. It is expected that rapid and inexpensive site evaluation procedures will eventually coalesce into methods to aid in the

evaluation of such characteristics as air-pollution potential or suitable locations for wind-powered electrical generator systems.

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Appendix II.—Notation

The following symbols are used in this paper:

h	=	base to peak height of hill or mountain;
L	=	half width of hill height, i.e., distance from hill peak to location where height is $1/2\ h$;
R	=	ratio of wind speed at height of 40 m to wind speed at height of 10 m;
u	=	wind speed;
α	=	wind speed velocity profile power law exponent; and
SLOPE	=	vertical hill rise expected in specified horizontal distance.

Empirical Models of Transit Service Areas

By Jerome M. Lutin,[1] Matthew Liotine,[2] and Thomas M. Ash[3]

(Reviewed by the Urban Transportation Division)

Introduction

It is known that the means of getting to and from transit systems and the ease or difficulty with which that portion of the journey is made can affect the traveler's decision to use transit, as much as do conditions and service on the system itself (6). That portion of a journey which is spent on the transit vehicle or waiting at stops is known as the *line-haul* portion of the trip. Those portions of the journey spent in getting to the transit system from the trip origin, and to the destination from the transit system, are known as the access-egress portions of the trip, or simply the *access* portion. Thus, accessibility to transit or transit access, deals with characteristics of the trip portions not on the transit system.

Planning for transit access is becoming more of a concern for transit planners. Given the impact of access characteristics on ridership, and the need for a normative definition of the service area accessed by transit, the development of a methodology to evaluate and plan for access is being undertaken. This research is directed towards the establishment of empirical tools for planning access to transit systems.

Defining Transit Service Areas

That portion of the urban area from which a transit line derives its patronage is known as its *service area.* No universal quantitative definition of a transit service area can be given, because its limits are not fixed, except by the habits (actual or expected) of the transit patrons. For the purposes of this work, a service area will be defined as an area bounded by a line representing the

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locus of all points of equal travel time or distance via a given access mode from a transit stop or station. As shown in Fig. 1, for a single transit station, typically located at a street corner on a rectangular street grid, the expected service area can be approximated by a diamond. For a web-type pattern of streets which converge at a station, a star or circle may be used to approximate

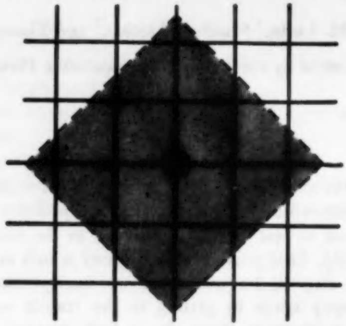

FIG. 1.—Service Area for Station on Rectangular Grid Access Network

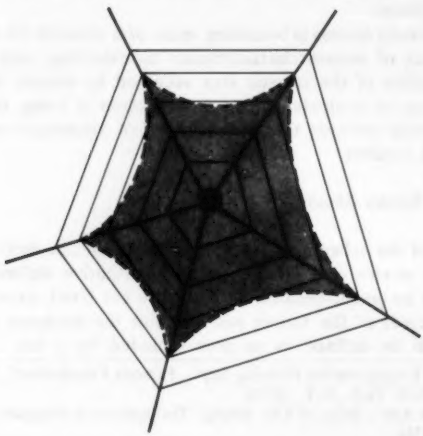

FIG. 2.—Service Area for Station on Converging Street Network

the service area boundary, as in Fig. 2 (2). For local bus services, the dominant mode of access is walking, and most bus lines stop on nearly every block along the route. For this type of service, one can assume an access service area comprised of overlapping areas which can be approximated by a constant width corridor along the transit line, as in Fig. 3. By drawing service area

boundaries as aforementioned, one can say that the population, locations, or trip ends falling within the boundaries are accessible by transit, and that the accessibility level is measured by the distance, or time from the transit stop to the boundary, or both.

The service area diagrams shown in Figs. 1–3 should be regarded with some caution for use by planners. First, these service areas may not represent a true measure of time or cost with respect to specific destinations. Second, they may not take into account the effects of competing stations, and third, they may not take into account the fact that transit trips volumes originating at an enroute station are generally weighted more in one direction than the other. For an analysis of the impacts of the foregoing variables, see Ref. 1. However, for many planning applications, the preceding service area models will be adequate representations.

A service area centered on a transit stop or transit line varies in radius according to the characteristics of the line-haul mode, mode of access, and socio-economic characteristics of the population to be served. In practice, service area boundaries can be described: (1) Empirically—as the inclusive boundary for the *x*th percentile

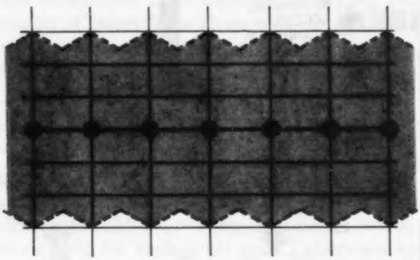

FIG. 3.—Service Area for Local Transit Route

of origins and destinations observed for patrons using a stop; or (2) normatively—as the arc of maximum distance for convenient or desirable travel to the transit stop. The terms "tributary area" or "commuter shed" have also been used to describe the service area. Two of the most important questions facing transit planners are: (1) How far from the transit line does one draw the service area boundary?; and (2) What is the relationship between this distance and some standard of desirable transit accessibility?

Three problem areas in transit planning have been identified for which access planning information is required: equity consideration, market penetration, and demand estimation for line-haul transit and access modes.

Equity Considerations

A transit stop or line is generally limited to serving the population adjacent to it, except for those lines which interface with other, regional transportation systems. Undoubtedly, planning the location of a new line or stop will exclude certain segments of the population. Determining who is and is not served by

the system is a major problem for the planner. To do this, the planner must first define access standards, stating the desired access time or distance radius. Next, he or she must determine the number of trip origins and destinations lying within that radius. Then, he or she must calculate the number of trip origins and destinations served both within and without the service area. This process, shown in Fig. 4, is repeated for each alternative route or stop under consideration, and the results compared, until an optimal service level is attained.

Since the population of an area to be served is not homogeneous, the planner may wish to stratify this analysis by population subgroup, examining service provided to minorities, the elderly population, and other groups. Definition of access standards may likewise be stratified. Pedestrian access standards for the elderly, e.g., may be shorter than those for the population as a whole. The most difficult part of the process is likely to be the definition of standards,

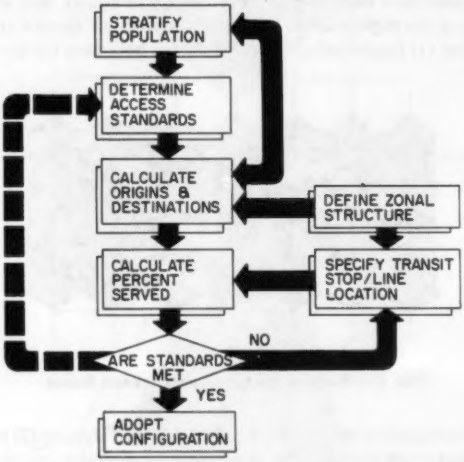

FIG. 4.—Transit Access Service Evaluation Process

since no accepted standards exist. The remaining tasks are relatively straightforward computations, with level of difficulty dependent upon the amount of data to be processed.

Market Penetration

To accurately predict transit patronage, the planner must estimate the magnitude of the population to be served, thus, the extent of the service area. This population, to whom transit service is available, represents the potential market for the transit line. The extent to which transit is selected for specific journeys by this population is the market penetration of transit. Estimation of market penetration requires knowledge of competing modes in the area, and a measure

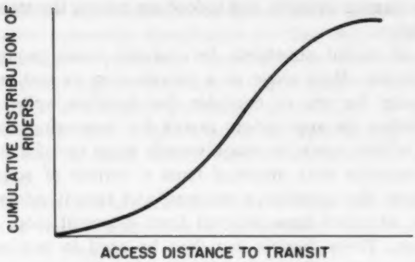

FIG. 5.—Cumulative Distribution of Riders as Function of Access Distance

of the disutility encountered in using each mode. In estimating market penetration, the planner should use an empirical description of the service area, rather than a normative standard. This is due to the behavioral nature of the choice process. To estimate market penetration from empirical data, one could calculate the cumulative distribution of riders for a given type of transit service by distance as in Fig. 5. Using this curve, one could estimate the maximum area adjacent to a similar transit line from which one could expect to attract patrons.

Demand Estimation for Line-Haul Transit

Analytical planning studies for new transit systems center around the development of patronage forecasts, the modal split for the new line. Most typically, the modal split is forecast by the use of a trip interchange model, which examines and determines mode split for each zonal pair. Calibration and application of mode split models require data on costs and times for various components of the trip, for each mode, and each zone pair. For the line-haul portion of each trip, these costs and time variables can be obtained from existing software, such as the Urban Transportation Planning System (UTPS) (3). However, the access times and costs are largely determined exogenously to the model system used, and often are only crude approximations. Since it is widely held that time spent getting to transit is more onerous than time spent onboard a transit vehicle, the planner should obtain an estimate of access time which is as accurate as possible.

Development of Models of Transit Service Areas

The objective of this research is to develop a concept which could be used by planners to determine a set of service area standards. These standards could be applied to existing and proposed transit systems to determine the proportion of the urban area served by a transit system. It is known that many factors, such as destinations served, transit travel time, frequency of service, hours of service, fare, security, reliability, accommodations for handicapped, and comfort, must be considered in judging whether transit service is available to an individual. However, access distances and service areas should be included

among system evaluation criteria, and indeed are among the most basic indicators of transit availability.

Development of useful standards for service areas requires answering a fundamental question. How close to a transit stop or station should a given location be in order for one to consider that location well-served by transit? One must also define the appropriate metric for measuring closeness. It is not within the scope of this report to unequivocally state specific standards. Rather, it is desired to examine data obtained from a variety of sources and present models which show the cumulative percentile of transit riders included within a given distance, or travel time interval from a transit stop, as derived from observed behavior. These models can then be used in two ways. First, such a model could be used to determine, for a given location at a distance from a transit line, the percentile access distance score for that location. Second, given a desired percentile score to be used as a normative access standard, the model can be used to find the radius which can be used to determine the normative service area boundary for that transit line. In this mode of operation, the model could be used to calculate the maximum, median, or any other distance according to the desired percentile of riders served.

In order to insure that accurate and useful models would be developed, it was decided to stratify models by three modes of access: (1) Pedestrian; (2) park and ride; and (3) kiss and ride. Within these three classes, models were further stratified by the line-haul mode. For pedestrian access, models were estimated for the local bus service in urban and suburban contexts, and for the express bus. For auto access modes, models were estimated for the commuter rail and the express bus, with express bus models estimated for service from both remote and peripheral parking lots. All pedestrian and commuter rail models were estimated on the basis of distance only, with distance expressed in feet in the former, and miles in the latter. All express bus with auto access models were estimated for both distance in miles and time in minutes. The aforementioned stratifications were limited by the availability of data. Consequently, no urban rail transit models could be calibrated due to the lack of suitable data.

Empirical Data and Modeling Process

The models presented are developed from access travel distance data (access mode: walk) for bus routes in Vancouver, British Columbia, Washington, D.C., and St. Louis, Mo., and from access travel distance, or access travel time data (access mode: auto) for commuter rail and express bus service in northeastern New Jersey, or both. Models for seven combinations of access and transit modes are offered: (1) Walk to urban bus; (2) walk to suburban bus; (3) walk to express bus; (4) park-and-ride to commuter rail; (5) park-and-ride to express bus; (6) kiss-and-ride to commuter rail; and (7) kiss-and-ride to express bus.

Data representing the first three combinations were derived from Refs. 4, 5, and 8. The data used to model park-and-ride to commuter rail, and park-and-ride to express bus, are derived from access distributions around six representative commuter rail stations and four representative express bus stops in northeastern New Jersey. The access distributions were computed from the data collected in surveys conducted by the Port Authority of New York and New Jersey between 1974 and 1976.

The curves presented represent access distributions around transit stops in terms of a cumulative percentile distribution or "less than" ogive. For a given access mode, a cumulative percentile distribution is constructed by summing the percentages of transit riders whose access trip originated within each distance or time interval. The cumulative percentile distribution is not a means of determining access modal split; it shows rather what percentage of transit patrons using the access mode, y, made access trips of less than the access distance, f, or the access time, t.

By inspecting the data, it was determined that nonlinear models would provide more explanation of the variance than linear models. However, there were no compelling theoretical reasons to favor one particular nonlinear model over another. Thus, a family of eight alternative model specifications was proposed. An interactive curve-fitting program was written which permitted one to select a data subset, and pick one of the model specifications. The program transformed the models into linear form and solved for the parameters using a least-mean squares regression technique. Statistics of R^2 and the standard error of estimate (SEE) were computed for fitted models in the nonlinear form, using untransformed variables. The interactive curve-fitting program produced scatter plots of the data with the fitted model curve superimposed, as shown in Figs. 6–19. Models were chosen through an iterative process, testing alternative forms and selecting the equation which produced the highest R^2 and lowest SEE. It should be noted, however, that the use of a least-squares regression for fitting models which have been linearized by taking logarithms may not produce the best estimates of model parameters. A generalized maximum likelihood approach is recommended (7).

Access Distributions—Walk to Bus Stop

These data were first categorized into "local urban and downtown" bus routes and "local urban" bus routes. The first category included one downtown Washington, D.C. bus route, which has a cumulative ridership percentile curve much steeper than the other urban routes used. The higher percentiles found in this route are plausible, since the downtown Washington, D.C. route was the only central business district (CBD) route, where densities and transit route competition are greater than in "urban area" routes. Elimination of the downtown D.C. route produced a better curve fit and decreased variance.

Fig. 6 shows the cumulative ridership percentile curve which best models pedestrian access for bus stops in urban areas *outside* the CBD. The equation of this curve is

$$Y_{pu} = 3.417 + 0.09739f - 0.00002324f^2; \quad f \leq 2095 \qquad (1)$$

in which Y_{pu} = cumulative ridership percentile at f for pedestrian access to local urban bus routes; and f = distance between origin and bus stop, in feet.

Fig. 7 presents the cumulative ridership percentile curve for local bus routes in suburban areas which produced the most plausible fit. The corresponding equation of this curve is

$$Y_{ps} = 118.0 \quad \exp\left(\frac{-448}{f}\right) \qquad (2)$$

in which Y_{ps} = cumulative ridership percentile at f for pedestrian access to suburban local bus routes. It should be noted that when the data from both

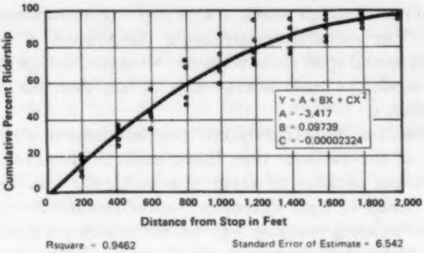

FIG. 6.—Cumulative Ridership Percentiles: Pedestrian Access Distribution for Local Urban Bus Stops

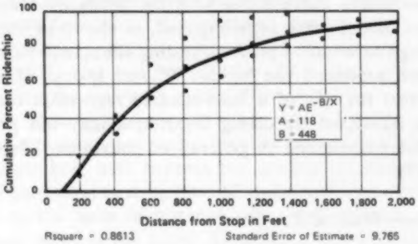

FIG. 7.—Cumulative Ridership Percentiles: Pedestrian Access Distribution for Local Suburban Bus Stops

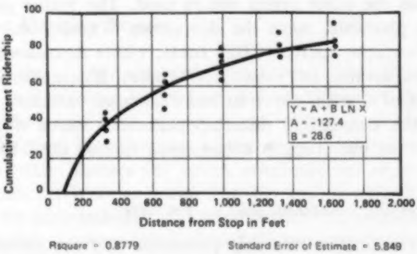

FIG. 8.—Cumulative Ridership Percentiles: Pedestrian Access Distribution for Express Bus Stops

the local urban and local surburban routes were combined, the resulting curves produced reasonable fits, but increased the variance in the data.

Fig. 8 presents the cumulative ridership percentile curve that best fitted the

data for pedestrian access to express bus routes. The corresponding equation is

$$Y_{pc} = -127.4 + 28.6 \ln f \quad \text{(3)}$$

in which Y_{pc} = cumulative ridership percentage at f for pedestrian access to express bus.

Automobile Access to Commuter Rail Stations

The plots in Figs. 9–11 show automobile access distributions, both park-and-ride (auto driver to station/stop) and kiss-and-ride (auto rider drops at station/stop), to commuter rail stations in the northeastern New Jersey area. All of the routes examined are radial in nature, centering upon the Newark/Jersey City/New York City metropolitan areas. For automobile access, the data is presented for both access distance and access time (measured in miles and minutes, respectively). As before, curves were then fitted to describe and to predict cumulative percentages of ridership originating within given distances or times from the transit interchanges. Only two functions, the negative exponential and

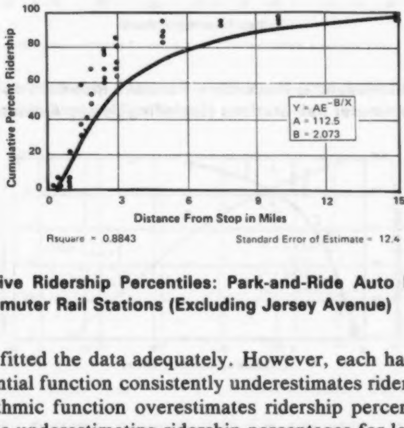

FIG. 9.—Cumulative Ridership Percentiles: Park-and-Ride Auto Driver Access Distribution for Commuter Rail Stations (Excluding Jersey Avenue)

the logarithmic, fitted the data adequately. However, each has drawbacks. The negative exponential function consistently underestimates ridership percentages, while the logarithmic function overestimates ridership percentages for shorter access trips while underestimating ridership percentages for longer access trips. Based solely upon the R^2 and SEE criteria, the negative exponential function, shown in Fig. 9, was selected as the best model for park-and-ride access to commuter rail service. The corresponding equation is:

$$Y_{ar} = 112.5 \exp\left(\frac{-2.073}{d}\right) \quad \text{(4)}$$

in which Y_{ar} = cumulative ridership percentile at d for park and rider; and d = driving distance between origin and rail stop, in miles.

The cumulative ridership percentages for car pool passenger travel time to rail stops, while not exactly analogous to auto driver data, were expected to closely resemble auto driver characteristics. In fitting the car pool passenger

time data, the logit curve produced the most reasonable distribution, shown in Fig. 10. The equation of this curve is

$$Y_{cr} = \frac{100}{1 + 19.11 \exp(-0.211t)} \quad \text{(5)}$$

in which Y_{cr} = cumulative ridership percentile at t for car pool passengers; and t = driving time between origin and rail top in minutes.

Fig. 11 plots cumulative ridership percentages for driving distances to rail stops for kiss-and-ride patrons. Only one equation, the negative exponential,

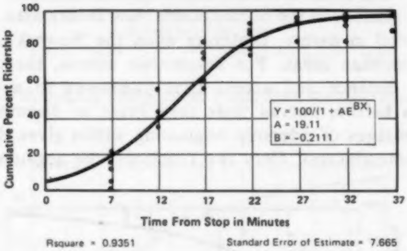

FIG. 10.—Cumulative Ridership Percentiles: Park-and-Ride Car Pool Passenger Access Distribution for Commuter Rail Stations (Excluding Jersey Avenue)

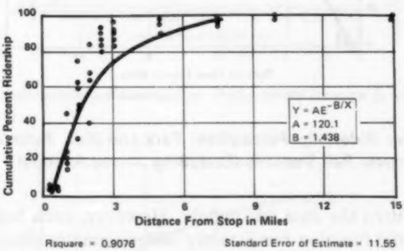

FIG. 11.—Cumulative Ridership Percentiles: Kiss-and-Ride Auto Distribution for Commuter Rail Stations (Excluding Jersey Avenue)

was able to fit these data acceptably. The negative exponential curve produces an R^2 of .9076, and an SEE of 11.55. However, like earlier equations, it overestimates short access trips and underestimates longer access trips. The model is

$$Y_{ka} = 120.1 \exp\left(\frac{-1.438}{d}\right) \quad \text{(6)}$$

in which Y_{ka} = cumulative ridership percentile at d for kiss-and-ride patrons; and d = driving distances between origin and rail stop, in miles.

Express Bus Stops: Characteristics and Access Distributions

Automobile access patterns to four express bus stops with substantial parking facilities are also examined. Two of these lots, the Vince Lombardi parking facility located in Jersey City at the entrance to the Lincoln Tunnel, and the North Bergen Township facility, are located across the Hudson River from Manhattan and can be classified as peripheral parking facilities. The two remaining lots are located in New Brunswick and in East Brunswick, N.J. Each lot is about 45 min away from New York City, and both are classified as remote lots.

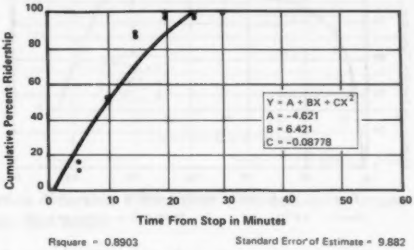

FIG. 12.—Cumulative Ridership Percentiles: Park-and-Ride Auto Driver Access Time Distribution for Remote Express Bus Stops

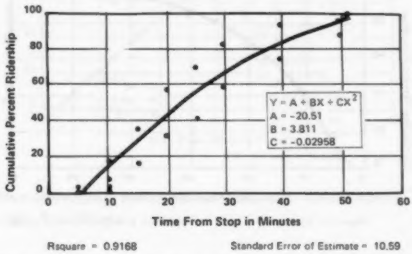

FIG. 13.—Cumulative Ridership Percentiles: Park-and-Ride Auto Driver Access Time Distribution for Peripheral Express Bus Stops

A comparison of data for the remote lots with those for the peripheral lots showed that automobile access patterns to remote lots and those to peripheral lots are dissimilar. Trips to the remote lots are on the whole much shorter than trips to the peripheral lots due to the choice of mode for the line-haul portion of the trip. Peripheral lot use indicates that the bus is chosen not as a line-haul mode, but more as a means of egress from the auto to the CBD. It is no surprise then, that the automobile trip to a peripheral lot is generally longer than an automobile trip to a remote lot. The automobile access patterns for kiss-and-ride to the same four express bus stops again showed that trips to remote lots are shorter on the whole than trips to peripheral lots. It is interesting

to see that the weighted mean access time for kiss-and-ride users to peripheral lots is longer than that for park-and-ride users. This result differs from the rail access patterns. One possible explanation is that people having destinations in Manhattan are being dropped off at the peripheral lots by people having destinations in the Jersey City area. This type of activity would occur much less frequently at rail interchanges.

Figs. 12 and 13 plot cumulative percentage curves for park-and-ride driving times to remote and peripheral express bus stops. Both curves fit the data adequately, although the data base is very small. The remote lot auto driver

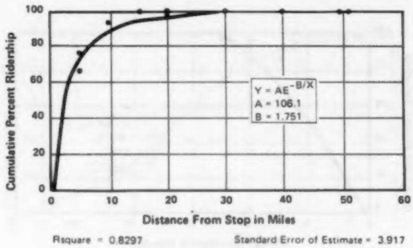

FIG. 14.—Cumulative Ridership Percentiles: Park-and-Ride Auto Driver Access Distance Distribution for Remote Express Bus Stops

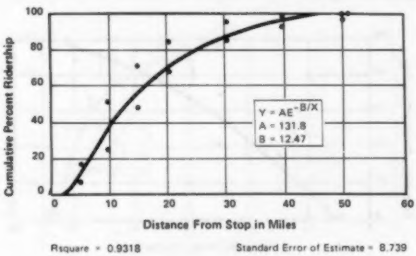

FIG. 15.—Cumulative Ridership Percentiles: Park-and-Ride Auto Driver Access Distance Distribution for Peripheral Express Bus Stops

curve is steeper than the peripheral curve; however it behaves unacceptably approximately beyond 25 min from the stop. At this value, the slope of the quadratic curve becomes negative, which results in decreasing cumulative percentiles, an obvious impossibility. In both cases, the quadratic function was found to best fit the data, producing R^2 of 0.8903 and 0.9168 and SEE of 9.882 and 19.59, respectively. The quadratic equation for auto driver access distribution by driving time for remote express bus stops, $Y_{ae'}$, is

$$Y_{ae'} = -4.621 + 6.421t - .08778t^2; \quad t < 24.5 \quad \quad (7)$$

The quadratic equation for auto driver access distribution by driving time for

peripheral express bus stops, $Y_{ac'}$, is

$$Y_{ac''} = -20.51 + 3.8116 - 0.02958t^2; \quad t < 55.7 \quad (8)$$

Curves modeling auto driver access distribution by driving distance for remote and peripheral express bus stops are presented in Figs. 14 and 15. As with the curves modeling driving times, curves modeling driving distances to remote lots are steeper than curves modeling driving distances to peripheral lots. In

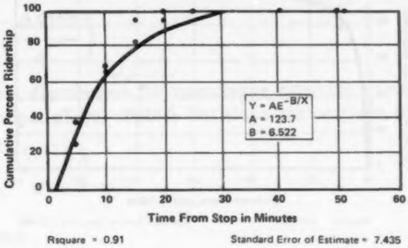

FIG. 16.—Cumulative Ridership Percentiles: Kiss-and-Ride Access Time Distribution for Remote Express Bus Stops

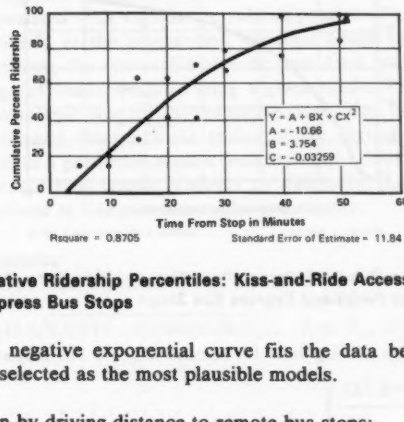

FIG. 17.—Cumulative Ridership Percentiles: Kiss-and-Ride Access Time Distribution for Peripheral Express Bus Stops

both cases, the negative exponential curve fits the data best. The following equations were selected as the most plausible models.

1. Distribution by driving distance to remote bus stops:

$$Y_{ae'} = 106.1 \exp\left(\frac{-1.751}{d}\right) \quad (9)$$

2. Distribution by driving distance for peripheral bus stops:

$$Y_{ae''} 131.8 \exp\left(\frac{-12.47}{d}\right); \quad (10)$$

$R^2 = 0.9318$; SEE $= 8.739$

Figs. 16 and 17 show the access distribution by driving time for kiss-and-ride passengers to remote and peripheral express bus stops. In these cases, the negative exponential function fits the remote lot data best, while the quadratic fits the peripheral data best. The negative exponential form produced an R^2 of 0.91 and a SEE of 7.435, while the quadratic equation produced an R^2 of 0.8705 and a SEE of 11.84. The equation chosen to model auto drop kiss-and-ride

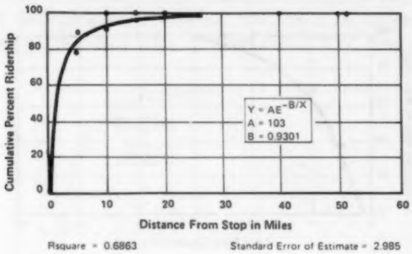

FIG. 18.—Cumulative Ridership Percentiles: Kiss-and-Ride Access Distance Distribution for Remote Express Bus Stops

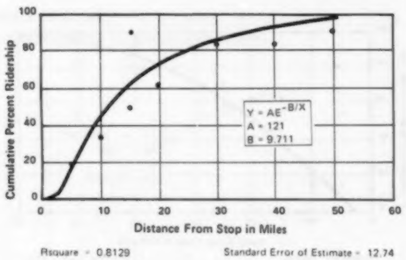

FIG. 19.—Cumulative Ridership Percentiles: Kiss-and-Ride Auto Drops Access Distance Distribution for Peripheral Express Bus Stops

access distribution by driving time for remote express bus stops is

$$Y_{ke'} = 123.7 \exp\left(\frac{-6.522}{t}\right) \quad (11)$$

The equation modeling kiss-and-ride access distribution by driving time for peripheral express bus stops is

$$Y_{ke''} = -10.66 + 3.754t - 0.03259t^2; \quad t \leq 57.6 \quad (12)$$

The final two equations model the kiss-and-ride access distribution by driving distance to remote and peripheral express bus stops. The curves produced from these equations are shown in Figs. 18 and 19, respectively. Once again, the curve modeling the distribution of access distances to remote lots is steeper

than the curve modeling the distribution of access distances to peripheral lots. As was the case with the access distributions around commuter rail stations, the curves modeling the access distribution of kiss-and-ride patrons are steeper than the auto driver curves. Similar to the equations modeling kiss-and-ride access distributions by time, the negative exponential curve fits both the remote lot data and the peripheral lot data best. Cumulative ridership percentiles for kiss-and-ride access to remote express bus stops, $Y_{ke'}$, can be estimated for distance using Eq. 13

$$Y_{ke'} = 103.0 \exp\left(\frac{-0.9301}{d}\right) \quad (13)$$

Kiss-and-ride access distribution for cumulative ridership percentiles, $Y_{ke''}$, by driving distance for peripheral express bus stops can be given by

$$Y_{ke''} = 121.0 \exp\left(\frac{-9.771}{d}\right) \quad (14)$$

Application of Models

For a given access mode and an access distance, the planner can use the calibrated models to estimate the percentage of transit patrons originating within the given access distance using the particular access mode. In Eqs. 15–28, the models are reformulated with cumulative ridership percentile, y (as a percent rather than a decimal), as the independent variable. These expressions enable the planner to estimate the access distance or time from within which comes a given percentage of transit patrons using a given mode. The median access distance or time to a transit stop for a given access mode, e.g., is easily estimated. Also, the planner could determine the radius of the service area, f, d, or t, which corresponds to a particular market penetration for ridership. These types of calculations would help transit planners to determine the level of service which a transit system or line provides to the community.

1. Pedestrian Access:
   (a) Distance to local urban bus stops outside CBD:

$$f = 2095.3 - 21515\sqrt{0.00917 - 0.00009296\, Y_{pu}}; \quad 0 \le Y_{pu} \le 98.6 \quad (15)$$

   (b) Distance to local suburban bus stops:

$$f = \frac{-448}{\ln Y_{ps} - 4.771}; \quad 0 < Y_{ps} \le 100 \quad (16)$$

   (c) Distance to express bus stops:

$$f = \exp\left(\frac{Y_{pe} + 127.4}{28.6}\right); \quad 0 \le Y_{pe} \le 100 \quad (17)$$

2. Park-and-Ride Access:
   (a) Distance to commuter rail stations:

$$d = \frac{-2.073}{\ln Y_{ar} - 4.723}; \quad 0 \leq Y_{ar} \leq 100 \; \ldots\ldots \; (18)$$

(b) Time to commuter rail stations:

$$t = 13.98 + 4.737 \ln \left(\frac{Y_{cr}}{100 - Y_{cr}} \right); \quad 0 < Y_{cr} < 100 \; \ldots\ldots \; (19)$$

(c) Distance to remote express bus stops:

$$d = \frac{-1.751}{\ln Y_{ae'} - 4.664}; \quad 0 < Y_{ae'} \leq 100 \; \ldots\ldots \; (20)$$

(d) Time to remote express bus stops:

$$t = 36.57 - 5.696 \sqrt{39.61 - 0.3511\, Y_{ae'}}; \quad 0 \leq Y_{ae'} \leq 100 \; \ldots\ldots \; (21)$$

(e) Distance to peripheral bus stops:

$$d = \frac{-12.47}{\ln Y_{ae''} - 4.881}; \quad 0 < Y_{ae'} \leq 100 \; \ldots\ldots \; (22)$$

(f) Time to peripheral bus stops:

$$t = 64.42 - 16.90 \sqrt{12.097 - 0.1183\, Y_{ae''}}; \quad 0 \leq Y_{ae''} \leq 100 \; \ldots\ldots \; (23)$$

3. Kiss-and-Ride Access:

(a) Distance to commuter rail stations:

$$d = \frac{-1.438}{\ln Y_{kr} - 4.788}; \quad 0 \leq Y_{kr} \leq 100 \; \ldots\ldots \; (24)$$

(b) Distance to remote express bus stops:

$$d = \frac{-0.9301}{\ln Y_{ke'} - 4.635}; \quad 0 \leq Y_{ke'} \leq 100 \; \ldots\ldots \; (25)$$

(c) Time to remote express bus stops:

$$t = \frac{-6.522}{\ln Y_{ke'} - 4.818}; \quad 0 < Y_{ke'} \leq 100 \; \ldots\ldots \; (26)$$

(d) Distance to peripheral express bus stops:

$$d = \frac{-9.711}{\ln Y_{ke''} - 4.796}; \quad 0 < Y_{ke''} \leq 100 \; \ldots\ldots \; (27)$$

(e) Time to peripheral express bus stops:

$$t = 57.59 - 15.34 \sqrt{12.70 - 0.1304\, Y_{ke''}}; \quad 0 \leq Y_{ke''} \leq 97.4 \; \ldots\ldots \; (28)$$

in which Y_c = cumulative ridership percentile for mode/access mode combination, c; f = walking distance, in feet, between origin/destination and bus stop at y; d = driving distance, in miles, between origin and transit stop at y; and t = driving time, in minutes, between origin and transit stop at y.

Conclusions

The accuracy and applicability of these models is restricted by the limitations on the data from which these models were constructed. The data, and thus, the models, do not make explicit impacts of station or stop competition, street patterns around the stop, the ridership habits of the stop's patrons, the socio-economic status of the stop's patrons, downtown parking rates, or highway congestion. The only independent variables used in the models presented are transit access trip distance and transit access trip time.

These models describe access distributions around an average stop for a given combination of access mode, transit mode, and urban location. They are therefore applicable on a system-wide or area-wide basis. As such, these models will be useful in estimating overall transit service market penetration. These models will not aid in making specific route location decisions, or in estimating specific tradeoffs between shorter line-haul times and shorter access times.

Models were constructed for all combinations of transit mode and access mode on which data were obtained. These were: (1) Walk to local bus service in an urban location; (2) walk to local bus service in a suburban location; (3) walk to express bus service; (4) park-and-ride to commuter rail service; (5) park-and-ride to express bus service; (6) kiss-and-ride to commuter rail service; and (7) kiss-and-ride to express bus service. Data were not obtained on subway service or feeder bus service.

Of the combinations examined, some produced better-fitting models than others. All of the models describing automobile access distributions for express bus stations were hampered by a lack of data once the data were divided into remote lot data and peripheral lot data. Thus, although curves were derived which fit the data well, the automobile access distributions for express bus service models are suspect.

Perhaps the best models were those describing walking distance to local bus service in urban locations, walking distance to express bus service, and auto rider driving times to commuter rail service. Each of these models exhibited a high R^2 and a SEE under 8%, which means that each describes the data well and has good predictive capabilities.

It is clear, however, that the models are accurate enough to determine normative classifications for transit service areas on the basis of access distance and time. It is expected that future research and data collection could yield even better estimates. In using these models, it is the individual planner who must make the crucial decisions about the appropriate standards to use.

Acknowledgment

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Data Collection via Satellite for Water Management[a]

By William G. Shope[1] and Richard W. Paulson[2]

(Reviewed by the Aerospace Division)

Introduction

The United States Geological Survey is operating a nationwide network of more than 9,000 automated hydrologic data-collection sites. The Geological Survey also operates an extensive computerized teleprocessing network used for data publication and data distribution to the water-user community. With the exception of several hundred sites, the data are transferred from the collection network to the teleprocessing network using manual operations at 4 week–6 week intervals. In an effort to decrease the time between collection and distribution, the Survey has been investigating the use of automated satellite telemetry to improve methods for remote-data acquisition. The objective of the improved remote-data acquisition system is to provide water-data users information in a timeframe that better meets water-data users needs. Automated remote-data acquisition also benefits the Survey through near real-time monitoring of instrument operation and hydrologic events. Where data are needed on a weekly or monthly basis and for sites that are difficult to access, remote-data acquisition through modern telemetry also results in data collection cost savings because of fewer visits to the data-collection sites.

The investigations into satellite telemetry by the Survey is a part of an overall instrumentation improvement program being conducted to improve and develop new automated sensors, data sampling techniques, data recording procedures, and data communication methods.

One of the major objectives of the telemetry investigations is to identify the requirements for real-time hydrologic data and how the data are used in water management. From this effort the Survey is formulating quantitative cost

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and benefit estimates for acquiring and distributing real-time hydrologic data. Through a series of experiments, demonstrations, and projects over the past 8 yr, the Survey has been evaluating satellite data-collection technology and has been attempting to improve and apply this technology to collect hydrologic data. On the basis of these ongoing tests, the Survey is developing the management procedures for the operation and monitoring of a real-time data-collection system that includes both Geological Survey and contractor personnel and services.

Most of the 200 Geological Survey data-collection sites instrumented for satellite communications are now fully operational with all or part of the costs shared by the water-data users. Efforts to collect economic and technical data are continuing, and will be intensified through a 105-site pilot project with COMSAT General Corporation. The use of brand names in this report is for identification purposes only and does not imply endorsement by the United States Geological Survey. The United States Geological Survey and COMSAT General Corporation 18-month pilot that began in August 1980, will permit the evaluation of a contracted service for real-time data collection. The program is also being used by Geological Survey economists to investigate how real-time data can be used in improving water-resources management and what benefits result.

In addition to the pilot contract service, the Survey is attempting to improve methods for delivery of real-time data to users through procurement of satellite direct-data readout stations and to improve instrumentation for the collection and transmission of data to the satellites.

Water Data Collection

Effective management of water resources requires that current hydrologic data be readily available to planners and managers. The Geological Survey has the principal responsibility within the Federal Government for providing hydrologic data and appraising water resources to facilitate evaluation of water problems. To accomplish its mission, the Survey, in cooperation with State and local governments and other Federal agencies, conducts investigations, surveys, studies and research on the occurrence, quality, quantity, distribution, utilization, movement, and availability of the nation's surface and ground-water resources. Essential to all of the studies are hydrologic data, and a most important activity of the Survey is its systematic, nationwide program of data collection, analysis, and dissemination.

The backbone of the Survey's collection of hydrologic data is an automated 9,000-site network used for collecting hydrologic data from streams and rivers throughout the United States. Data from these stations provide information for a wide variety of uses such as: (1) Planning; (2) managing reservoirs; (3) issuing flood warnings; (4) allocating water for irrigation and hydroelectric power; (5) monitoring the flow of streams to insure that treaties, compacts, and other legal agreements are honored; and (6) monitoring the quality and quantity of the water in the nation's rivers and streams.

At the present time, measurements taken at most data-collection stations are automatically punched on paper tapes that are retrieved during visits to each site at intervals of 4 weeks–6 weeks. The tapes are manually checked for time and data reading errors and then entered into devices that transmit the data through telephone lines to the Survey's national headquarters in Reston, Va.,

where the data are recorded on computer tapes. After the data have been edited and processed by the Survey computers, they are available for retrieval through the Survey's national network of remote computer terminals. Although the measurements are recorded on site in a timely manner, the collection and processing under this system includes a time lag of at least 4 weeks–6 weeks because the data are manually retrieved from the stations.

The Geological Survey national computer network consists of four large computers in the Survey's national and regional headquarters, and approx 250 remote computer terminals throughout the United States. The hydrologic data entered into this network are processed and stored in the Survey's National Water Data Storage and Retrieval System (WATSTORE). Water data stored in WATSTORE, as well as several data bases outside the Survey are indexed by the National Water Data Exchange (NAWDEX). It is through NAWDEX and the WATSTORE system that the majority of hydrologic data are disseminated to the water-user community through computer information retrievals and publications. Additional information on WATSTORE can be found in Showen (6), and information on NAWDEX can be found in Edwards (2).

The procedures for acquiring data from the collection sites and entering the data into WATSTORE are manpower intensive and time consuming. These two factors are causes for concern in terms of present and future demands for more and timely data. National economic growth has meant an increase in the number of users of hydrologic data who operate hydropower generating plants and irrigation systems and who manage flood damage prevention and navigational waterways. Growth has also generated interest from users such as urban planners who must consider the hydrologic system in their planning, and environmentalists who want to clean up or prevent further environmental degradation. These additional requirements for hydrologic data have been translated into a need for additional manpower, into a need for more hydrologic data, particularly water-quality parameters, and into a need for these data to be delivered to users in a more timely manner.

Telemetering data from automated data-collection sites resolves other problems besides saving manpower and acquiring more timely data. The operating agency has available a link to their remote sites in order to monitor hydrologic conditions and instrumentation performance in order to schedule special field visits to collect unique samples, take measurements, or repair the data-collection equipment. More timely data from the sites through remote-data acquisition can mean better data-collection network management.

User requests for more timely data, data with a smaller elapsed time between its observation and receipt, has been met only partially. The time requirement for accessing data has been diminished by the paper-tape recording devices, but the time required for getting the tape to the computer is constrained by the frequency of retrieval of the tape from its stream-recording site. Requirements of some users for more timely data have been satisfied by increased site visits for pickup and provision of data every 30 days, and for others every 10 days, but this provision has been costly in terms of manpower. For several applications, water data are needed several times daily or even hourly, such as in optimally allocating the available water supply between irrigation and hydropower generation, or in optimizing water releases from a dam system in order to minimize flood damages. Where demand for water-resource data has been on a real time

basis (hourly or daily), the Geological Survey has attempted to respond by installing a variety of telemetry systems based on land lines or line-of-sight radio networks. In some instances where real-time hydrologic data needs have been vital, the responsible water-data user agency, has installed and operated telemetering devices in the Survey's river gaging stations. This has resulted in a growing number of local independent water-data telemetry systems. These independent telemetry systems have often proved costly because of the inherent redundancy between systems and the need for installation and maintenance of relay stations (radio) and cables (land lines). The reliability of these systems has also proven to be a problem especially during severe weather conditions when water-data needs are most critical.

Remote Data Telemetry

The telemetry of data from remote data-collection sites can and is being accomplished by a variety of mechanisms. The major components of a telemetry system are the sensors that measure or detect changes in an observed constituent such as stream stage, encoders that convert the sensor output to forms suitable for transmission, a transmission system that provides the link from a sensor to another location, and a data reception and distribution facility that sorts, decodes, checks, and distributes the incoming data. The most important component, and the one that characterizes the entire system, is the transmission media. Conventional methods include land lines (telephone) and high-frequency and ultrahigh-frequency (microwave line-of-site) radios. Exterterrestrial methods applied within the last 10 yr include meteorburst and satellite data-collection systems.

The reader is referred to Halliday (3), for a more complete review of telemetry systems.

The selection of a telemetry system for an application calls for careful consideration of many factors such as cost, reliability, responsiveness, coverage, growth, and flexibility. The choice of a telemetry system for an area of limited size and uniform geographic characteristics could result in any of the conventional or exterterrestrial systems best meeting the users needs. The Geological Survey's objective is to develop a national system that will meet the needs of a variety of water users and data collectors under a wide range of geographic conditions. A series of tests that began in 1972 with the National Aeronautical and Space Administration's Landsat satellite data-collection system (DCS), continue today with the geostationary orbital environmental satellite GOES, to evaluate satellite telemetry for the Geological Survey's national hydrologic remote data-acquisition system. The GOES satellites are operated by the National Earth Satellite Service (NESS) of the National Oceanic and Atmospheric Administration (NOAA). The tests conducted by the Geological Survey included two Government operated satellites (Landsat and GOES) and a commercial satellite used in an 8-month demonstration with COMSAT General Corporation and Telesat of Canada. For more complete descriptions of the tests, the reader is referred to Carter and Paulson (1). The results of the tests have shown Satellite DCS, when compared with other telemetry systems, to be generally cost-effective, reliable, easy to install and operate, able to cover vast areas, and extremely flexible in carrying a wide variety of data transmissions that can be scheduled to meet user needs.

Satellite Data Collection

Satellite data collection systems are built around Earth orbiting satellites that are used to relay data transmissions from networks of data collection sites to one or more receive sites. The onboard instrumentation that performs the relay function is a satellite transponder. The transponder is a combination radio receiver and transmitter which receives data messages and instantaneously retransmits the data back to Earth without any alteration except for frequency and signal strength. A satellite data collection system is made up of five major elements that include: (1) Sensors; (2) small radios called data-collection platforms; (3) satellites; (4) Earth receiving site(s); and (5) a data processing and distribution system. The sensors used by the Geological Survey are generally battery operated and include an onsite 16-channel paper-tape recording device that encodes the data and provides an interface to the data-collection platforms (DCP). The paper tapes are used as a backup in case of failure of the telemetry system. The

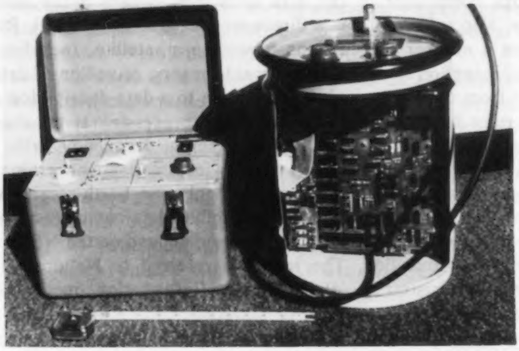

FIG. 1.—Cutaway View of GOES Data Collection Platform (DCP) and Test Set used to Program and Test DCP

DCPs also are battery operated (12 volt) and are generally positioned adjacent to the sensor-recorders and provide the sensor control and radio transmissions to the satellite. The DCPs offered in today's market place offer a wide range of capabilities that include both reception of command messages relayed from a control center via the satellite (interrogation) and self-timed transmission of buffered blocks of data at fixed intervals. Many of these DCPs can also transmit random-emergency messages immediately following the detection (by the DCP) of a critical element of hydrologic data. The major components of a DCP are the power unit (battery with solar panels for recharge), antenna, computer memory, programmable microprocessors, clock, and radio. Costs of these DCPs generally range between \$2,500–\$5.000 for a complete system. A cutaway view of a DCP and the test set used to program and test the DCP are shown in Fig. 1.

In order for a DCP to successfully transmit to a satellite, it must have a

clear line-of-sight to the space craft. To complete the link back to Earth, the satellite must simultaneously also have a clear line-of-sight (view) to the receive sites. The view of each satellite is determined by its orbit and there are basically two types of orbits used. Satellites placed in an orbit coincident with the Earth's equatorial plane at about 36,000 km, rotate at the same speed as the Earth, and appear to be in a fixed position. These geostationary or geosynchronous satellites have a constant view of almost a full hemisphere. The GOES satellite and Telesat's ANIK satellite, that was used in a commercial communications-satellite demonstration with COMSAT General, are geosynchronous satellites.

Another type of satellite is a polar-orbiting spacecraft that circles the Earth every 100 min–110 min at altitudes of approx 900 km. Because of the orbiting characteristics of these types of satellites, the spacecraft has a mutual view of a DCP and receive site, only 2%–3% of a 24-h day for most locations in the lower 48 States. Examples of the polar-orbiting satellites are the Landsat and NOAA series (TIROS-N and NOAA-6). A more complete coverage of DCPs and satellites can be found in Carter (1).

The fourth component of the data collection system is the satellite Earth receive site, also referred to as a direct-readout ground terminal. Receive sites can perform a number of functions related to a satellite, including command and control, reception of data from onboard sensors, reception of data messages transmitted from the DCPs, and dissemination to a data-distribution system.

An important function of the data-message processing at the receive site is the monitoring of DCP-message signal strength, frequency drift, and other parameters that give the operator valuable information concerning the entire data-collection system. The NESS operates only one Earth receive site located at Wallops Island, Va., for the GOES satellite data-communications system. Most users of the system, including the Survey, depend on the NESS centralized system for data acquisition. The reader is referred to National Oceanic and Atmospheric Administration (4) for detailed information on the GOES data-collection system.

Several users of the GOES system in an effort to reduce system complexity, increase timeliness of the data, and reduce dependence on land lines, are turning to local passive (receive only) satellite direct-readout ground terminals. These terminals, which can be obtained for $80,000–$120,000, provide a user with immediate access to the data transmitted from the satellite.

Included in the last element of a satellite data-collection system are data processing and distribution. This part of the system provides a means to deliver the data to the user and convert the data to information that will assist the water-resources manager in making decisions and taking needed actions. Many of the major components of the satellite data-collection system are operational, however, the processing and distribution of real-time data is not yet fully defined or automated. The Geological Survey is now working with the users of real-time hydrologic data in order to identify the requirements of an automated data-distribution system. There are two operational satellite data collection systems in use today. The GOES and TIROS-N-ARGOS are providing satellite data-relay service to a wide variety of national and international users. In both systems, the data-collection agency must install and operate the sensors and data-collection platforms. The NESS provides and operates the spacecraft and Earth receiving facilities and also provides limited data distribution. Data processing, storage,

and distribution to the water-data users is the responsibility of the data-collection agency.

Applications

The Geological Survey is operating an expanding 210-site network (August 1980) that is supported by data relay through the GOES satellites. These sites are operated by Government employees with equipment purchased by the Survey. The GOES satellite data communication system is provided at no cost to the Survey under a Memorandum of Agreement with NESS. This agreement specifies NESS as the operator of the satellites, Earth receive site, and a data-distribution center. The Survey, through a project office in Reston, Va., operates a communications minicomputer that acquires the DCP messages from the NESS data-distribution center in Camp Springs, Md. After acquisition of hydrologic data from NESS, the communications minicomputer automatically enters the data into the Survey's *HY*drologic *D*ata *RE*al-time *C*omputer Processing *S*ystem (HYDRECS) located on the Geological Survey's Reston, Va., Honeywell Multics computer. Data are entered into HYDRECS to provide water-data users with 24-h access on demand, to real-time hydrologic data.

Users access hydrologic data from the computer in an interactive mode, permitting almost immediate retrievals of hydrologic data that were transmitted only minutes earlier from the national network of telemetered remote sites. The HYDRECS system functions as an experimental real-time front end for the National Water Data Storage and Retrieval System (WATSTORE). At the end of each day, data are automatically transferred to the WATSTORE system located in the Geological Survey's Amdahl computer.

By early 1981, the Geological Survey will be adding an additional 105 sites to its network through a contract with COMSAT General Corp. This pilot contract requires the vendor to provide a real-time data-collection service, including the installation, maintenance, and operation at existing Geological Survey data-collection sites of all equipment required to measure and transmit data to the GOES satellite. The COMSAT General will also operate a direct-readout terminal and data processing and distribution system. The pilot effort will permit the Survey the opportunity to evaluate a contractor service in comparison with ongoing Survey operated activities. If costs permit, a contractor effort may be one alternative for expanding real-time data-collection activities in situations where manpower is not available.

Of the 210 sites now operated by personnel from the Geological Survey, all except 9 are paid for in part or in whole by the water-data users. The biggest single user is the United States Army Corps of Engineers who depend on real-time hydrologic data for reservoir management. The Geological Survey has been collecting hydrologic data for the Corps and many other Federal agencies for many years. The addition of satellite telemetry to monitor gate openings in dams, reservoir levels, and tailwater and headwater conditions is an enhancement of the existing program. The Corps estimates that close to 2,000 sites may be instrumented for satellite telemetry over the next 5 yr. Many of these sites are targeted to be operated in cooperation with the Geological Survey.

Another major application for real-time hydrologic data is the collection and analysis of rainfall and stream-stage values for flood warning. The Mount St.

Helens eruption on May 18, 1980, caused major changes in the hydrology of Spirit Lake and rivers north and west of the volcano. A large flood followed the major eruption, and there are fears that additional floods may follow. To monitor the hydrologic conditions near Mount St. Helens, 10 data-collection platforms with capability for immediate transmittal of emergency messages have been placed near the mountain at strategic locations. The emergency messages will be transmitted should the river stage exceed a specific limit programmed into the platforms by the operator during installation. During stable conditions, these platforms will transmit data at regularly scheduled 3-h intervals. The platforms are operated by the Geological Survey's office in Tacoma, Wash.

The Tacoma office also operates data-collection platforms that collect hydrologic information for users concerned with hydroelectric power generation. In addition to the GOES telemetry, the Tacoma office currently operates a 40-site ultrahigh-frequency line-of-site terrestrial radio network for telemetering hydrologic data from key locations. In 1981, Tacoma will begin replacing this

FIG. 2.—GOES Hydrologic Data Collection Site in Florida Everglades

network with a satellite relay network built around a local GOES direct-readout ground terminal. This terminal will be shared by other Survey offices in the Pacific northwest and will include other Federal agencies. The Tacoma terminal will serve as a prototype activity for future terminals that may be used in other Survey field offices.

A second application related to flood warning is a 40-site GOES satellite telemetry network for collection of rainfall and stream-stage data in the Salt River basin in Arizona. The first 23 sites are scheduled to be operating in 1980, with the remaining sites to start operation in 1981.

In all applications related to the collection of data for flood warning. The principal data user is the river forecast center network of the National Weather Service (NWS). These centers collect and analyze all relevant information including data from the GOES telemetry system and data from the NWS observation systems prior to issuing their river forecasts and flood warnings. Hydrologic data from all GOES data-collection platforms are automatically made available to the NWS computers in Suitland, Md. through internal NOAA

communication links between NESS and NWS. These same computers are used for running the river forecast models used by the majority of NWS river forecast centers.

There are a number of other applications for real-time hydrologic data that could significantly benefit water-data users in managing water resources. These applications include monitoring water quality, water supply forecasting, managing navigational waters, allocating urban water supplies, and managing water resources for irrigation. Data for these and other water management applications are being telemetered from Geological Survey data collection sites, but the data relayed via satellite are not yet fully utilized in a real-time mode (within an hour of data sensing).

The primary utilization of satellite telemetry in the Geological Survey today is operation and management of portions of the hydrologic data-collection network. Because of manpower and transportation costs, it is often less expensive to invest in and operate a satellite telemetry system than to make frequent visits to remote sites to collect data and inspect the instrumentation. Sites that are difficult to access or require provisional data to be distributed more often than the normal 6-week manual acquisition cycle are good candidates for satellite telemetry. Fig. 2 shows a GOES site in the Florida Everglades where stage and rainfall data are collected and transmitted. Sites such as this one often require a helicopter for access. In addition to reducing the time requirements for entry of data into the WATSTORE system from 4 weeks–6 weeks, to a minimum of 1 day, the field operations personnel are also able to monitor the performance of automated instrumentation, including the data-collection platforms, and to observe in relative real time, the occurrence of critical or unusual hydrologic events. These events may warrant unscheduled site visits in order to manually collect water-quality samples or measure the stream discharge.

Economic Studies

The costs of starting and operating a satellite data-collection system and the impact and application of these systems on the national hydrologic data-collection network are well understood. What is not yet fully known is the future requirements and potential benefits to society of a data-collection system that can provide timely data to the water-resources management community. In order to identify and document these requirements and potential benefits, the Geological Survey has initiated a number of economic studies concerned with the availability of real-time hydrologic data. The pilot program with COMSAT General Corporation is being conducted primarily to document the benefits, costs, uses, and requirements for providing real-time hydrologic data to the water-data user community. The Geological Survey completed an initial benefit-cost study in November, 1978. Based on the projected benefits from flood warning and irrigation-water management, funds were allocated to conduct the pilot program (1980–1982) to verify the projected benefits and to study potential benefits derived from the availability of real-time data monitoring. The new economic study is to be completed in early 1982 and will attempt to quantify benefits and costs through actual water-data user involvement with real-time hydrologic data. The economic study is also being used to analyze the cost effectiveness of satellite

data collection. The initial benefit-cost study determined that for at least 90% of the Survey sites operational cost savings for data collection will *not* outweigh the added current costs of procuring and installing a satellite data-collection system. With increased future use of the satellite for data collection, and improved technology, the costs for satellite telemetry are expected to decrease in relation to station operation costs.

For the last 2 yr the Geological Survey's satellite telemetry network has been growing at an average of 60 sites per year. Accelerated growth of this network is now dependent on the 1982 economic study and the relationship of costs to the benefits derived from the use of real-time hydrologic data.

Development Activities

The development of an effective remote-data telemetry system is an on-going activity that may require 3–4 additional years before a fully operational national system is in place. At the present time, almost 75% of efforts of the Geological Survey related to satellite telemetry are devoted to planning, research and development. A very brief summary of these activities are presented in the following:

1. Economic studies of costs, benefits, uses, and requirements.
2. Pilot test of a contractor service.
3. Interagency coordination through several working groups attempting to improve, standardize, and promote the use of GOES.
4. Data processing improvements to provide further automation of data distribution including user alert systems.
5. Instrumentation improvement to include:
   —Standardized data-collection platforms.
   —Intelligent sensors that are able to vary recording, measuring, and communication frequencies.
   —Development of sensors to directly measure parameters that now require visits to the sites.
6. Test local satellite direct-readout ground terminals.
7. Develop a management strategy for the operation of a national network of sites utilizing satellite telemetry.

Summary and Conclusions

The Geological Survey is operating large data-collection and teleprocessing networks for acquiring and distributing hydrologic data to the water-data user community. Efforts are being made to bridge the gap between the collection and teleprocessing networks with an automated telemetry system that can provide hydrologic data in a timeframe that will help improve the management of the Nation's water resources. Real-time hydrologic data are potentially useful in a number of applications that include improved flood warnings, irrigation water allocations, water supply forecasting, reservoir management, monitoring water quality, hydropower generation, managing navigational waters, and allocating urban water supplies.

An economic study conducted by the Geological Survey forecasts the accumu-

lated benefits to be derived through availability of real-time data for flood warning and irrigation-water allocation, outweigh the costs of providing the satellite telemetry system required to acquire the data. The same study shows that manpower savings for operations alone are too small to justify the costs incurred in developing and operating the major part of the telemetry system.

The Geological Survey is continuing to study the costs and benefits of providing real-time hydrologic data to the water-data user community. The contracted 105-site COMSAT General pilot project will be used to study the improvements to the management of water resources provided by the availability of real-time hydrologic data.

Satellite data telemetry has already proven to be a reliable, and for many applications a cost-effective tool for the remote acquisition of hydrologic data. The Geological Survey through 8 yr of experimentation, contractor services and studies, new instrumentation development, and intergovernmental-agency cooperative programs is attempting to improve satellite data-collection methods. Over 200 sites now operated by the Survey are fully operational with costs paid for in full or in part by the water-data users. Justification for satellite telemetry at these sites is based on special requirements such as site accessibility or special data reporting requested and financed by the water-data users. Sites with these special characteristics make up a small percentage of the existing 9,000 site network. A significant expansion of the existing satellite telemetry network now depends on a more complete understanding of the needs for real-time hydrologic data for water-resources management and the additional benefits to be derived from such use.

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Node Flow Analysis of Water Distribution Systems

By Pramod R. Bhave[1]

(Reviewed by the Pipeline Division)

Introduction

Water distribution systems are used in many branches of engineering. They are used for urban and rural water supply purposes, industrial water supply schemes, and sprinkler irrigation projects. The present design practice for such water distribution systems is to provide an economical system that can supply the design demands at the various nodes at heads at least equal to the minimum required ones. However, in due time, it may happen that the actual demands at some nodes may be more than the design demands due to reasons such as: (1) The actual growth pattern may be different from the anticipated one; or (2) the system outlives its design life. In such cases, the distribution system, as it is, may prove to be inadequate, at least at some of the nodes, to satisfy the nodal demands. A looped system may be inadequate, even for design demands, when some pipes are closed for cleaning, repairs, or replacement. To determine whether a distribution system is adequate to satisfy the demands, an analysis of the distribution system is carried out.

In the distribution system analysis, it is assumed that the demands at the various nodes are satisfied and accordingly the flows in the various links and the corresponding nodal heads are estimated. This type of analysis usually deals with the estimation of nodal heads and therefore is herein termed Node Head Analysis (NHA). [As stated by Shamir and Howard (11), occasionally, the consumptions at some of the nodes may be unknown and therefore required to be estimated. However, even this type of analysis can be considered as NHA as shown later. Thus, the NHA is the usual solution of the pipe network analysis in which either a flow condition or a head condition is imposed at each node.] In the NHA, the nodal heads are estimated and then compared with the required ones. Thus, the NHA helps in locating the nodes that are deficient in head requirements, in estimating the nodal head deficiency, and also in estimating the required additional heads at the source nodes or the

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boosting pressures in the system to ensure the adequate nodal heads.

In the NHA it is assumed that the additional heads at the source nodes or the boosting pressures are available and therefore the nodal supplies are equal to the nodal demands. However, if such additional heads or boosting pressures are not available, the distribution system fails to simultaneously provide the demands at all the nodes. Though some nodes may be able to satisfy the demands, some others may partially fulfill the demand requirements while the remaining may fail to meet any demand at all. To analyze such a situation it will be necessary to consider the distribution system as it is and then estimate the actual outflows available at the various nodes and the discharges in the various pipes. In contrast to the NHA, this type of analysis is herein termed Node Flow Analysis (NFA). Thus in the NFA it is assumed that: (1) Neither any additional heads at source nodes nor any additional boosting pressures anywhere in the distribution system are provided; and (2) flow at a node can occur only if the available nodal head is at least equal to the minimum required one. Assumption 2 implies that no consumption can occur at a node unless the minimum required pressure exists. A valve like behavior is assumed wherein the valve opens completely but only when sufficient pressure is available.

Several approaches are available for the NHA of distribution systems. The approaches are based on the Hardy Cross method (4), the Newton-Raphson method and its variations (5,6,8,10,11), the Newton-Cross method (7), linear theory (14), and electrical analyzers (9). Although the writer has developed NFA techniques for serial water distribution systems (1), no methodology is presently available for the NFA of the distribution systems in general. The purpose of this paper is to develop such a methodology.

Definitions

Some definitions are introduced herein. A *pipe* is an element of the distribution system that has a constant flow and no branches. A *node* is a point where two or more pipes meet, a pipe ends, or a pipe begins. One set of nodes, termed *source* (or inflow) *nodes* supplies water to the distribution system which in turn transmits it through the various pipes to another set of nodes, termed *consumption* (or outflow) *nodes*. (Even when the consumption at a node is zero, it is included herein in the set of consumption nodes. Thus all the nodes which are not source nodes are treated herein as consumption nodes.) When the head available at a node, H_j, is equal to the minimum required one, H_j^{min}, the node is a *critical node*. When the available head is more than the minimum required one, the node is a *supercritical node* and when it is less, the node is a *subcritical node*. When the demand at a consumption node is zero, the node is a *zero-demand node*. When a consumption node is unable to provide any demand, i.e., when the outflow at a consumption node, Q_j, is zero even though there is a demand, Q_j^{req}, the node is a *no-flow node*. When a consumption node partially fulfills the demand, it is a *partial-flow node*. When a consumption node actually supplies the required demand, it is an *adequate-flow node*. When a consumption node provides outflow even more than the required demand, it is a *surplus-flow node*. When the outflow at a consumption node is negative, it is a *negative-flow node*. In a distribution system, actually, no consumption node generally behaves either as a surplus-flow node or as a negative-flow

node. However, these terms are introduced herein as such situations occur, though only transitionally, in the NFA procedure suggested later.

Node-Category Compatibility

From the various definitions given earlier, it is seen that a node category depends upon the head and the outflow. When both these categories are considered together, the nature of the node can be completely described. However, in practice, a subcritical node cannot be an adequate-flow node, or a no-flow node cannot be a supercritical node. Thus, the two categories of a node can be combined together only when they are compatible with each other. Such compatibility of node categories is given in Table 1. As negative-flow nodes and surplus-flow nodes do not generally occur in practice, these categories are not included therein.

Theory

As stated previously, a distribution system may become inadequate to simultaneously meet the demands at all the consumption nodes. However, the distribution

TABLE 1.—Compatibility of Node Categories

Node category (1)	Compatible with node category (2)
Subcritical	Zero-demand, no-flow
Critical	Zero-demand, partial-flow (exceptionally no-flow or adequate-flow)
Supercritical	Zero-demand, adequate-flow
Zero-demand	Subcritical, critical, supercritical
No-flow	Subcritical (exceptionally critical)
Partial-flow	Critical
Adequate-flow	Supercritical (exceptionally critical)

system, when left to itself, will try to satisfy the demands as far as possible and therefore the flow in the system gets stabilized such that the total outflow from it is maximum under the given conditions. Thus the NFA problem can be looked upon as a system outflow optimization problem subject to certain constraints.

Problem Formulation.—Consider a distribution system having M source nodes denoted by m, $m = 1 \ldots M$, N consumption nodes (consumption can be zero) denoted by j, $j = 1 \ldots N$, and L elements denoted by l, $l = 1 \ldots L$. Let Q_m and Q_j denote the inflow at the source nodes and outflow at the consumption nodes, respectively. The objective function therefore becomes

$$\text{Maximize:} \quad \text{Total outflow} = \sum_{j=1}^{N} Q_j \qquad (1)$$

At all the source nodes the head, H_m, should be equal to the available head, H_m^{avl}. Thus

$H_m = H_m^{\mathrm{avl}}$ for all m . . . . (2)

A zero-demand consumption node can be subcritical, critical, or supercritical. However, for other consumption nodes the actual outflow will depend upon the node-category. Thus

$Q_j = Q_j^{\mathrm{req}}$ if $H_j > H_j^{\min}$ for all j . . . . (3)

$0 \le Q_j \le Q_j^{\mathrm{req}}$ if $H_j = H_j^{\min}$ for all j . . . . (4)

$Q_j = 0$ if $H_j < H_j^{\min}$ for all j . . . . (5)

in which Q_j^{req} = required demand at node j. Eqs. 3, 4, and 5 herein constitute a set of alternate constraints (13).

To preserve the flow continuity at each node, the algebraic sum of all the flows at a node must equal zero. Thus, if q_{ij} denotes the flow in pipe ij, from node i to consumption node j, then for all the consumption nodes

$$Q_j = \sum_j q_{ij} \quad \text{for all } j \quad \ldots\ldots \quad (6)$$

Similarly, if q_{im} denotes the flow in pipe im, from node i to source node m, for all the source nodes

$$Q_m = -\sum_m q_{im} \quad \text{for all } m \quad \ldots\ldots \quad (7)$$

The head loss in an element l, HL_l, may be expressed as

$\mathrm{HL}_l = \phi_l(q_l)$ for all l . . . . (8)

in which ϕ_l = some function of the discharge q_l in the element l. Functional form ϕ_l, which is mostly nonlinear, is not specifically defined herein and can represent a variety of elements, including simple pipes (l may be ij or im), minor loss elements such as valves (for which $\phi_l > 0$ when $q_l > 0$), and pumps (for which $\phi_l < 0$ when $q_l > 0$).

For looped distribution networks, the algebraic sum of all the headlosses along a loop must be zero. Thus

$$\sum_{\mathrm{loop}} \mathrm{HL} = 0 \quad \text{for all loops} \quad \ldots\ldots \quad (9)$$

The complete optimization problem can now be stated as

$$\text{Maximize: Total outflow} = \sum_{j=1}^{N} Q_j \quad \ldots\ldots \quad (10a)$$

Subject to

$H_m = H_m^{\mathrm{avl}}$ for all m . . . . (10b)

$Q_j = Q_j^{\mathrm{req}}$ if $H_j > H_j^{\min}$ for all j . . . . (10c)

$0 \le Q_j \le Q_j^{\mathrm{req}}$ if $H_j = H_j^{\min}$ for all j . . . . (10d)

$Q_j = 0$ if $H_j < H_j^{\min}$ for all j . . . . (10e)

$$Q_j = \sum_j q_{ij} \quad \text{for all } j \qquad (10f)$$

$$Q_m = -\sum_m q_{im} \quad \text{for all } m \qquad (10g)$$

$$HL_l = \phi_l(q_l) \quad \text{for all } l \qquad (10h)$$

$$\sum_{\text{loop}} HL = 0 \quad \text{for all loops} \qquad (10i)$$

It is seen that this optimization problem is a nonlinear one with certain constraints as alternate constraints.

Problem Solution.—The usual NHA of a distribution system is the simultaneous solution of Eqs. 2, 6, 7, 8, and 9. Collins, et al. (2,3) have recently shown that when the function ϕ_l is strictly monotone increasing (which holds for all the elements except certain pumps), the unique solution of Eqs. 2, 6, 7, 8 and 9 and thus the NHA of the distribution system is the same as the optimum solution of a network flow optimization problem. Conversely, therefore, the solution of a network flow optimization problem can be obtained by carrying out the NHA of the distribution system. However, the optimization problem formulated herein (Eq. 10) is different from that of Collins, et al. (2,3) because of the presence of the alternate constraints of Eqs. 3, 4, and 5. However, because well established procedures with computer programs are available for carrying out the NHA of a distribution system, the solution of the optimization problem of Eq. 10 and therefore the NFA of the distribution system is herein proposed to be obtained by repeatedly carrying out the NHA of the system as shown later.

Now, as stated by Shamir and Howard (12), the following rules have to be satisfied for the NHA of a distribution system:

1. A node having an unknown consumption (supply) should be connected to at least one other node with a known consumption (supply).
2. A subsystem consisting of an element with unknown characteristic and its two end nodes should have no more than one additional unknown—one of the heads or consumptions (supplies) at the two end nodes.
3. Considering any node, at least one of the following should be unknown: the consumption (supply) at the node; the head at the node itself or at any adjacent node; or the characteristic of an element that is connected at the node.
4. The total number of the unknowns must be equal to the total number of the nodes (supply as well as consumption) in the system.

When all the four rules are satisfied, the analysis of the distribution system is the NHA. Thus, even when consumptions at some of the nodes are unknown, as long as all the four rules are satisfied, the problem is herein treated as a NHA problem.

The situation, however, is different in the problem formulated herein. The characteristics of all the elements are known and therefore the question of satisfying rule 2 does not arise. Except for the consumption nodes where the

demand is zero, at all the remaining consumption nodes the actual outflows are unknown (only their limiting lower and upper values are known). The number of these unknowns therefore lies between 1 (the system must have at least one non-zero-demand node) and N (when all consumption nodes are non-zero-demand nodes). Further, at all the N consumption nodes the actual heads are also unknown. At the M supply nodes, though the heads are known, the actual supplies which depend upon the actual nodal outflows are unknown (total number M). Thus the total number of the unknowns lies between $M + N + 1$ and $M + 2N$, while the total number of the nodes and therefore the total number of the available equations is $M + N$. Thus rule 4 is not satisfied. It can be seen that though rule 3 is satisfied, rule 1 is not satisfied.

Solution Procedure.—The NFA of the distribution system is proposed to be achieved by repeatedly carrying out the NHA of the system. However, as the number of unknowns is more than the number of knowns, to make the NHA solution feasible, values of some of the unknowns are required to be assumed. For all the source nodes, the available heads are fixed and therefore known for all the iterations, while the actual supplies are unknown. For nonzero-demand consumption nodes either the head or the outflow is assumed for each iteration

TABLE 2.—Conversion of Node Categories

Obtained node category (1)	Convert to node category (2)
Negative-flow	No-flow
Partial-flow	Critical
Surplus-flow	Adequate-flow
Subcritical	Critical
Supercritical	Adequate-flow

so that there is also only one unknown for each consumption node and thus, the NHA of the system becomes feasible. The NHA solution thus obtained forms one iteration of the NFA solution.

The solution procedure consists of the following steps:

1. For the first NFA iteration, treat all the consumption nodes as adequate-flow nodes, obtain the NHA solution of the system, estimate the available heads at all the consumption nodes and estimate their head-dependent category (subcritical, critical or supercritical).
2. Refer to Table 1 and check the compatibility of the obtained node category with the assumed node category. If they are compatible for all the nodes, the solution of the optimization problem and thus the NFA of the distribution system is achieved. (For the first NFA iteration, as the assumed category for all the consumption nodes is 'adequate-flow,' the category compatible with this is 'supercritical.' Thus, if all the consumption nodes are found to be supercritical, the NFA is already achieved. In such a case the NFA and the NHA are the same.)
3. When the node-category compatibility is not achieved for a consumption node, convert the obtained node category to the one shown in Table 2. When

the node-category compatibility is achieved for a consumption node, retain the assumed (not obtained) node category.

4. Treat the converted or retained node category as the assumed category and carry out the next NFA iteration. Estimate the outflow or the available head as the case may be for each consumption node, decide its corresponding node category and then go to step 2.

Illustrative Example

The procedure for carrying out the NFA of a distribution system is illustrated herein with the help of a simple hypothetical distribution system (Fig. 1) having one source (labeled 1) and six consumption nodes (labeled 2-7). The system has no valves or pumps and the headloss for the pipe elements is given by $HL = rq^2$, the r values being as shown along the pipes in Fig. 1. The available

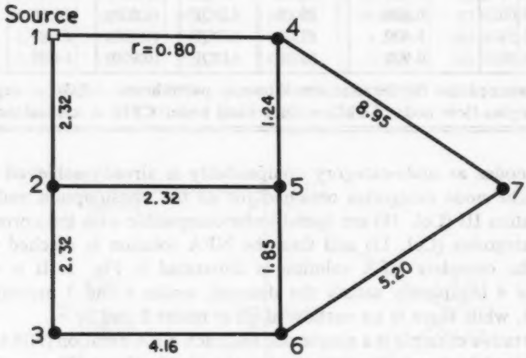

FIG. 1.—Distribution System for Illustrative Example (r values are for headloss in feet and discharge in cubic feet per second. Multiply by 380.8 to obtain r values for headloss in meters and discharge in cubic meters per second)

head at the source node and the minimum required head for the consumption nodes are given in Col. 2, Table 3, while the supply and the design outflows are given in Col. 3. The available heads at all the nodes for the design consumptions are given in Col. 4. It is seen that all the nodes are supercritical and the design is adequate.

The link 1-2 (Fig. 1) is now closed for repairs and the NFA solution of the distribution system is required. The NFA iterations are shown in Table 3. The values assumed for the iterations are shown in parentheses. All the consumption nodes are assumed as adequate-flow nodes (Col. 5) for iteration I, but all of them are found to be subcritical (Col. 8) and therefore converted to critical category (Col. 9). At the end of iteration II (Col. 12), it is seen that nodes 2 and 5 are negative-flow nodes, nodes 6 and 7 are partial-flow nodes, while nodes 3 and 4 are surplus-flow nodes. The negative-flow nodes 2 and 5 are converted to no-flow nodes, the surplus-flow nodes 3 and 4 are converted to adequate-flow nodes while critical nodes 6 and 7 are retained

TABLE 3.—Node

				LINK NFA			
				I			
Node (1)	Required head, in feet (2)	Consumption, in cubic feet per second (3)	Available head for design solution, in feet (4)	Assumed nature of node (5)	Outflow, in cubic feet per second (6)	Head, in feet (7)	Obtained nature of node (8)
1	100.00	−5.000	100.00	—	−5.000	(100.00)[a]	—
2	87.00	0.700	89.92	ADQF	(0.700)	63.51	SUBC
3	85.00	0.800	87.30	ADQF	(0.800)	63.01	SUBC
4	90.00	0.400	93.20	ADQF	(0.400)	80.00	SUBC
5	88.00	0.800	89.69	ADQF	(0.800)	66.66	SUBC
6	85.50	1.400	87.01	ADQF	(1.400)	63.47	SUBC
7	86.00	0.900	86.98	ADQF	(0.900)	64.39	SUBC

[a]Values assumed for the iteration are shown in parentheses. NEGF = negative-flow
SURF = surplus-flow node; SUBC = subcritical node; CRIT = critical node; SUPC

as critical nodes as node-category compatibility is already achieved for them (Col. 13). The node categories obtained for all the consumption nodes at the end of iteration III (Col. 16) are found to be compatible with the corresponding assumed categories (Col. 13) and thus the NFA solution is reached (Cols. 14 and 15). The complete NFA solution is illustrated in Fig. 2. It is seen that nodes 3 and 4 completely satisfy the demand, nodes 6 and 7 partially satisfy the demand, while there is no outflow at all at nodes 2 and 5.

This illustrative example is a simple one and each NFA iteration (NHA solution) can be obtained by hand calculation using the Hardy Cross, Newton-Raphson, or linear theory methods. However, as each iteration of the NFA solution is the NHA solution of the distribution system, NFA solution of large complex systems involving valves and pumps can be obtained by using a computer program based on any of the NHA methods. To obtain the NFA solution, either the computer programs are required to be extended or the input data modified at the end of each NFA iteration.

Although only three NFA iterations are required to obtain the NFA solution for the illustrative example, more NFA iterations will be necessary for large networks. However, it will be observed that the number of nodes, where node-category compatibility is achieved successively, increases after each iteration.

The NFA procedure, suggested herein, starts with an infeasible solution (except when the node-category compatibility is achieved at all the nodes at the end of first iteration), but at every NFA iteration, as the guiding constraints are different, a different solution is generated. However, at successive NFA iteration, the number of nodes for which node-category compatibility is achieved increases, and therefore the solution, though still infeasible, approaches towards the feasible solution. Thus, in the end, when the node-category compatibility is achieved for all the nodes, the alternate constraints of Eqs. 3–5 are properly satisfied

Flow Analysis Solution

1–2 CLOSED ITERATIONS							
II				III			
Assumed nature of node (9)	Outflow, in cubic feet per second (10)	Head, in feet (11)	Obtained nature of node (12)	Assumed nature of node (13)	Outflow, in cubic feet per second (14)	Head, in feet (15)	Obtained nature of node (16)
—	−3.535	(100.00)	—	—	−3.108	(100.00)	—
CRIT	−0.272	(87.00)	NEGF	NOFL	(0.000)	86.70	SUBC
CRIT	1.275	(85.00)	SURF	ADQF	(0.800)	85.48	SUPC
CRIT	1.597	(90.00)	SURF	ADQF	(0.400)	92.27	SUPC
CRIT	−0.549	(88.00)	NEGF	NOFL	(0.000)	87.93	SUBC
CRIT	1.126	(85.50)	PRTF	CRIT	1.381	(85.50)	PRTF
CRIT	0.358	(86.00)	PRTF	CRIT	0.527	(86.00)	PRTF

node; NOFL = no-flow node; PRTF = partial-flow node; ADQF = adequate flow node; = supercritical node. 1 ft = 0.305 m; 1 cfs = 0.0283 m^3/s.

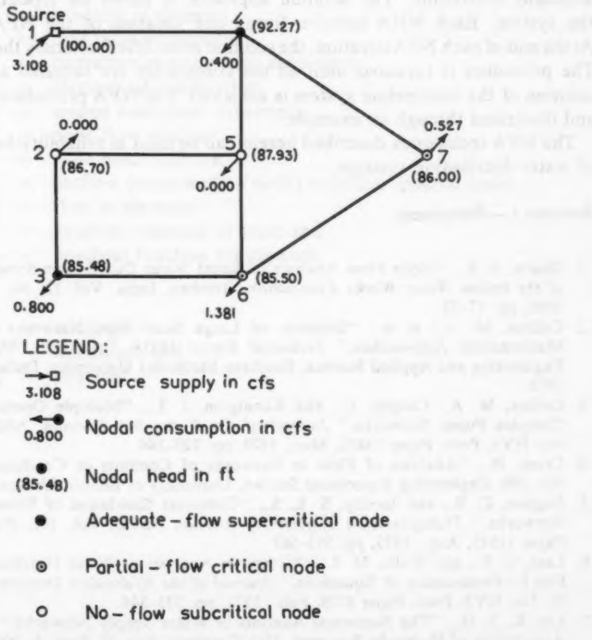

FIG. 2.—Node Flow Analysis Solution (1 ft = 0.305 m; 1 cfs = 0.0283 m^3/s)

and, therefore, the NFA solution and, thus, the optimal solution for Eq. 10 is attained.

No mathematical proof is given herein to show that the NFA solution is the optimal solution. However, it will be observed that a slight variation in the NFA solution will result either in a suboptimal solution or in an infeasible solution wherein one or more of the constraints are violated.

Summary and Conclusions

The presently available distribution system analysis techniques analyze the distribution systems assuming that all the nodal demands are satisfied, by providing, if necessary, additional head at the sources or boosting pressures in the distribution systems. This type of analysis is herein termed *node head analysis*. However, if no additional heads or boosting pressures are provided, the distribution system behaves differently. Although some nodes may be able to completely satisfy the demand, some others may satisfy the demand partially while the rest may completely fail and may not provide any demand at all. This type of analysis is herein termed *node flow analysis*.

The NFA of a distribution system is considered herein as a system outflow optimization problem consisting of linear and nonlinear constraints, some being alternate constraints. The solution approach is based on repetitive NHA of the system. Each NHA solution forms one iteration of the NFA procedure. At the end of each NFA iteration, the solution is modified to satisfy the constraints. The procedure is repeated until all the constraints are satisfied and the NFA solution of the distribution system is achieved. The NFA procedure is described and illustrated through an example.

The NFA techniques described herein can be used in reliability-based analysis of water distribution systems.

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Appendix II.—Notation

The following symbols are used in this paper:

H = head at node;
HL = headloss in element;
ij, im = pipe (also subscript);
$j = 1 \ldots N$ = consumption node (also subscript);
$l = 1 \ldots L$ = element (also subscript);
$m = 1 \ldots M$ = source node (also subscript);
NFA = node flow analysis;
NHA = node head analysis;
Q = outflow (consumption node) or inflow (source node);
q = flow in element;
r = headloss constant of pipe; and
ϕ = headloss function for element.

Superscripts

avl = available;
min = minimum; and
req = required.

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Buried Pipelines: Settlement Modification and Load Transfer

By John H. Scarino,[1] F. ASCE

(Reviewed by the Pipeline Division)

Introduction

As a consequence of numerous variable factors, such as depth of cover, soil properties, and construction conditions, buried pipelines generally are subject to unequal loading. Furthermore, the subgrade of pipelines is rarely uniform. These conditions will result in some nonuniformity of pipeline settlement, although there will be some smoothing effect, depending upon the pipeline's stiffness.

Welded or otherwise rigid jointed pipelines can be considered as beams on elastic foundations, and, as such, would provide a good smoothing system to counteract the effects of the heterogeneity of the ground and loading conditions. Bolotin (1) presents such an analysis for pipelines buried in a soil whose properties vary statistically. Such approaches have limited application, but should be further explored, particularly with regard to safety factor evaluations.

This paper does not consider the problem of the distribution of stochastic stresses and strains in connection with rigidly jointed underground pipelines, but limits itself to the settlement smoothing and load transfer effects of pipelines with flexible joints. For such pipelines, nonuniform settlement can result in angular distortion between individual lengths of pipe. This causes the joints to open and thus create a potential for leakage. Additionally, in the process of dampening settlements, shears can be transmitted across the joints, which would result in increases and decreases of the loadings and stresses of the individual pipe sections.

Assumptions

Initially, for derivation purposes, it is assumed that:

1. Buried pipelines are articulated structures whose joints are hinges that are 100% efficient in transmitting shear.

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2. Loads imposed on the pipelines remain constant, i.e., independent of the differential settlement resulting from shear transmittal.

3. Other than those adjacent to other structures, the individual pipe sections, herein referred to as links, are equal in length.

4. The pipe links are rigid in a longitudinal direction. Accordingly, contact pressure beneath each link can be computed using the fundamental principles of mechanics.

5. Load-related settlements are proportional to the contract pressures beneath each link.

These assumptions are evaluated in Appendix I to ascertain their validity and to determine their impact on the sensitivity of the results.

Derivation and Examples

A link, BC, as shown in Fig. 1, having a length of L and subjected to a uniform load of w per unit length, when subjected to an end shear of V, will

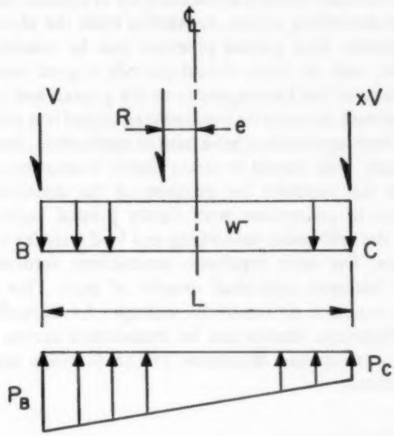

FIG. 1.—Link Subjected to Uniform Vertical Loading

be pushed downward at B increasing the contact pressure at B and reducing the contact pressures at C. Accordingly, settlement at C would be less than that caused by load w alone. However, the adjacent length of pipe subjected to the same uniform load will tend to settle, thus developing a reaction at C equal to xV, in which x = the shear transfer coefficient. The resultant, R, of these loads is equal to $wL + xV(1 + x)$ and the eccentricity, e, of this resultant is

$$e = \frac{VL(1 - x)}{2L\,[wL + V(1 + x)]} \quad \text{(1)}$$

In accordance with Assumption 4, and provided that the resultant falls within the middle third of the link, the contact pressures are

$$P_B = \frac{wL + V(1+x)}{L}\left\{1 + \frac{6VL(1-x)}{2L\,[wL + V(1+x)]}\right\} = w + \frac{2V}{L}(2-x) \quad . . . (2)$$

$$P_C = \frac{wL + V(1+x)}{L}\left\{1 - \frac{6VL(1-x)}{2L\,[wL + V(1+x)]}\right\} = w - \frac{2V}{L}(1-2x) . . . (3)$$

For the adjacent link CB the shear at joint C is equal to $-xV$. Substituting this value in Eq. 2, the contact pressure at joint C becomes

$$P_C = w - \frac{2xV}{L}(2-x) \quad . . . . (4)$$

by equating the contact pressures at each side of joint C, as shown in Fig. 2, the equation to determine the shear transfer coefficient is $x^2 - 4x + 1 = 0$, in which $x = 0.2679$.

The values of shear transfer coefficients at other joints are powers of x.

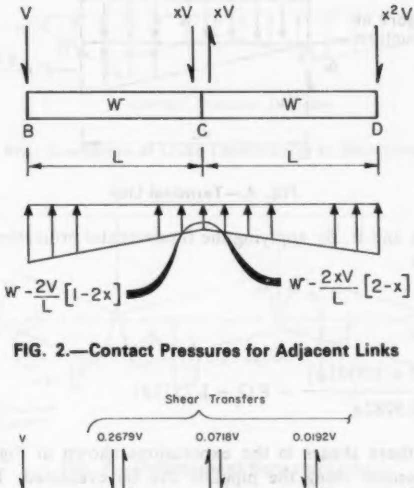

FIG. 2.—Contact Pressures for Adjacent Links

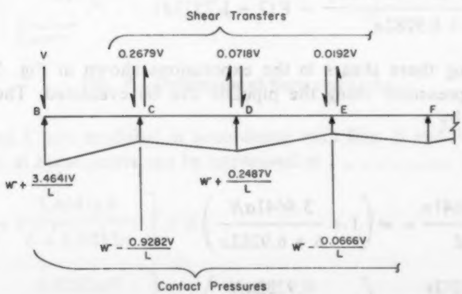

FIG. 3.—Joint Shears and Contact Pressures for Series of Links

The shear transfers and contact pressures that result when these are substituted in Eqs. 2 and 3 are shown in Fig. 3.

At the terminus of a pipeline, adjacent to a structure, various conditions can be assumed. Generally, the pipe section adjacent to the structure can be assumed to be suspended at the structure end and fully soil-supported at the far end. Fig. 4 shows this condition for link AB with a pipe section length equal to aL. Loading is assumed to vary, since, for positive projecting conduits, there is a complete projection condition at the face of the structure while at the far end an incomplete projection condition exists. In addition, the weight of the pipe and contents must be considered. The loading at joint B is $w_c + w_p + w_w$, which equals w; the loading at joint A is $N_c w_c + w_p + w_w$. If $N = (N_c w_c + w_p + w_w)/w$, then the total loading at joint A can be expressed as Nw. Assuming a joint shear of V at joint B, the contact pressure at joint B is assumed as equal to that of the adjacent pipe length, or equal to $w + 3.4641\,V/L$. Both the loading and contact pressures are considered to vary linearly

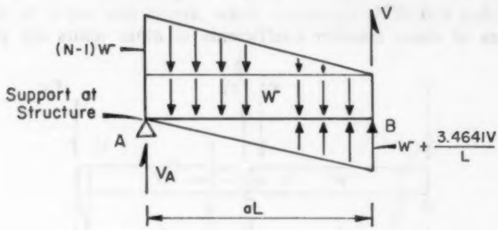

FIG. 4.—Terminal Link

between joints A and B. By applying the fundamental principles for equilibrium, the reactions are

$$V = \frac{w\,a\,L\,N}{6 + 6.9282a} \quad (5)$$

$$V_A = \frac{w\,a\,L\,N(2 + 1.7321a)}{6 + 6.9282a} = V(2 + 1.7321a) \quad (6)$$

By substituting these shears in the expressions shown in Fig. 3, joint shears and contact pressures along the pipeline can be evaluated. These are shown in Fig. 5 and by

$$P_A = 0 \quad (7)$$

$$P_B = w + \frac{3.4641v}{L} = w\left(1 + \frac{3.4641aN}{6 + 6.9282a}\right) \quad (8)$$

$$P_C = w - \frac{0.9282v}{L} = w\left(1 - \frac{0.9282aN}{6 + 6.9282a}\right) \quad (9)$$

$$P_D = w + \frac{0.2487v}{L} = w\left(1 + \frac{0.2487aN}{6 + 6.9282a}\right) \quad \text{(10)}$$

As noted earlier, the resulting nonuniform pressures cause unequal settlements which, in turn, will tend to open the pipe joints. The settlement of the pipe involves two components. The first element, denoted S_w, is due to soil compressibility and is taken as proportional to the contact pressure, w, in accordance with Assumption 5. The second element, denoted S_m, is the result of bedding adjustment and is not considered a function of loading. Since the contact pressures

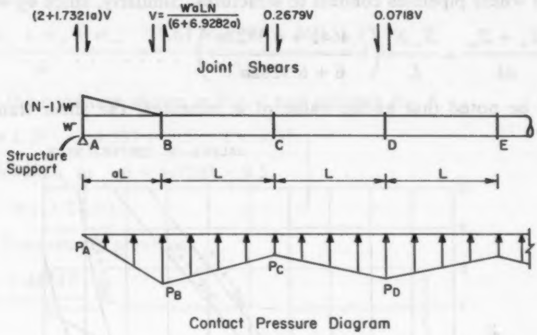

FIG. 5.—Series of Links Terminating at Structure

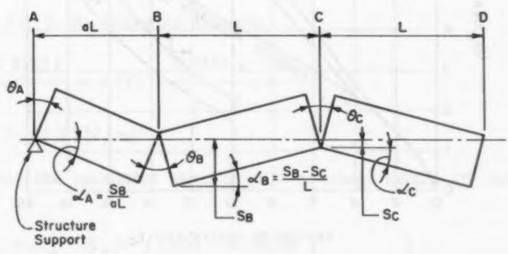

FIG. 6.—Deflection of Series of Links

at joints B and C are modified in accordance with Eqs. 8 and 9, respectively, the settlement at these joints can be expressed as

$$S_B = S_w\left[1 + \frac{3.4641aN}{6 + 6.9282a}\right] + S_m \quad \text{(11)}$$

$$S_C = S_w\left[1 - \frac{0.9282aN}{6 + 6.9282a}\right] + S_m \quad \text{(12)}$$

(See Fig. 6, which shows the links adjacent to a structure with their joints deflected.)

The angular deflection of links and angular joint openings are expressed in terms of known parameters. Since $\theta_B = \alpha_A + \alpha_B$, one can determine upon substitution and simplification that

$$\sin \theta_B = \frac{S_w + S_m}{aL} + \frac{S_w N}{L}\left(\frac{3.4641 + 4.3923a}{6 + 6.9282a}\right) \quad (13)$$

Then Eq. 13 can be used to check the maximum angular joint opening that will occur where pipelines connect to structures. Similarly, since $\theta_A = \alpha_A$

$$\sin \theta_A = \frac{S_w + S_m}{aL} + \frac{S_w N}{L}\left(\frac{3.4641 + 4.3923a}{6 + 6.9282a}\right) \quad (14)$$

It should be noted that as the value of a increases, the shear transfers and

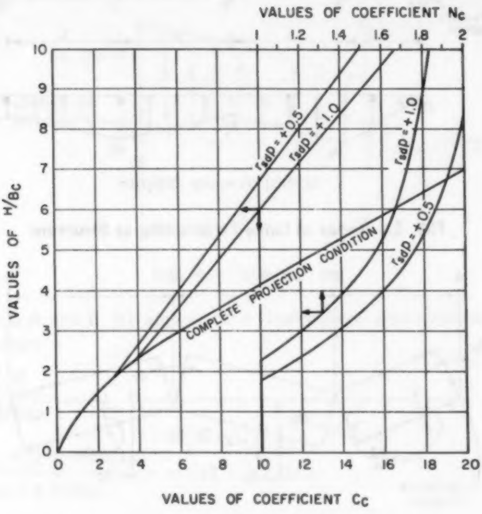

FIG. 7.—Diagram for Coefficients C_c and N_c

angular joint openings decrease, while the longitudinal bending in link AB will increase. (The determination of the optimal value of a is beyond the scope of this paper.)

The application of some of the derived equations is illustrated by the examples that follow.

Example A.—Consider a 24-in. (610-mm) diam reinforced concrete pipe laid in a projecting embankment condition with $r_{sd}p = +0.5$ and with

$B_c = 2.5$ ft (0.76 m); $S_w = 0.08$ ft (0.02 m);

$H = 10$ ft (3 m); $S_m = 0.02$ ft (0.006 m);

$\gamma = 110$ lb/ft^3 (180 kg/m^3); $w_p + w_w = 460$ lb/ft (6.71 kN/m);

$L = 8$ ft (2.4 m); $a = 0.5$ . . . . (15)

From Fig. 7 for $H/B_c = 10/2.5 = 4.0$, $C_c = 5.93$, and $N_c = 1.61$

$w_c = C_c w B_c^2 = 5.93 \times 110 \times 2.5^2 = 4{,}077$ lb/ft (59.5 kN/m);

$w = 4{,}077 + 460 = 4{,}537$ lb/ft (66.2 kN/m) . . . . (16)

By definition

$$N = \frac{N_c w_c + w_p + w_w}{w} = \frac{1.61 \times 4{,}077 + 460}{4{,}537} = 1.55 \quad \text{(17)}$$

Assuming the condition shown in Fig. 5 and utilizing Eq. 5

$$\left.\begin{aligned} V &= \frac{w\,a\,L\,N}{6 + 6.9282a} = \frac{4{,}537 \times 0.5 \times 8 \times 1.55}{6 + 6.9282 \times 0.5} \\ V &= 2{,}972 \text{ lb (13.2 kN)} \end{aligned}\right\} \quad \text{(18)}$$

With Eq. 8 maximum pressure

$$\left.\begin{aligned} P_B &= w + \frac{3.4641V}{L} \\ P_B &= 4{,}537 + \frac{(3.4641)(2{,}972)}{8} = 5{,}824 \text{ lb/ft (85.0 kN/m)} \end{aligned}\right\} \quad \text{(19)}$$

Also, using Eq. 9, the minimum pressure

$$\left.\begin{aligned} P_C &= w - \frac{0.9282V}{L} = 4{,}537 - \frac{0.9282 \times 2{,}972}{8} \\ &= 4{,}192 \text{ lb/ft (61.2 kN/m)} \end{aligned}\right\} \quad \text{(20)}$$

To determine the maximum angular joint opening, values are substituted in Eq. 13:

$$\left.\begin{aligned} \sin\theta_B &= \frac{S_w + S_m}{aL} + \frac{S_w N}{L}\left(\frac{3.461 + 4.3923a}{6 + 6.9282a}\right) \\ &= \frac{0.08 + 0.02}{0.5 \times 8} + \frac{0.08 \times 1.61}{8}\left(\frac{3.4641 + 4.3923 \times 0.5}{6 + 6.9282 \times 0.5}\right) \\ &= 0.03463 \\ \theta_B &= 1.98^\circ \text{ (3.46 crad)} \end{aligned}\right\} \quad \text{(21)}$$

To determine the required three-edge strength for pipe, it is suggested that the average pressure over a length equal to twice the pipe's outside diameter from the point of maximum pressure be used. For Example A, the average contact pressure over a two-diameter length adjacent to the maximum pressure

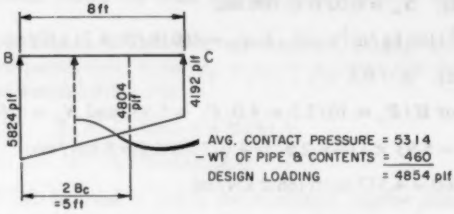

FIG. 8.—Design Loading Determination

is 5,314 lb/ft (77.6 kN/m), as shown in Fig. 8. Deducting the weight of the pipe and its contents, the design loading becomes 4,854 lb/ft (70.8 kN/m).

Example B.—Consider a buried pipeline subjected to concentrated surface loads, such as result from truck traffic or railroads. Two of the many possible positions for loads with respect to the pipe jointing are shown in Figs. 9 and 10. These are somewhat unique in practice, but are limiting cases.

Assume a link length of 8 ft (2.4 m); B_c = 2.5 ft (0.8 m); P = 10,000 lb (44.5 kN) (including impact) and H = 3 ft (0.9 m). Since $B_c/2H = 2.5/(2 \times 3) = 0.417$ and $L/2H = 8/(2 \times 3) = 1.333$; the load coefficient from Table XXVI of Ref. 2 is 0.445. The load on the pipe due to the concentrated load is

$$W_{sc} = C_s \frac{PF}{L} = 0.445 \frac{10{,}000 \times 1.50}{8} = 834 \text{ lb/ft } (12.2 \text{ kN/m}) \quad \ldots\ldots\ldots \quad (22)$$

Based on statics, the pressure under link AB in Fig. 9 is equal to $(w + W_{sc}) - 2V/L$, which equals the maximum pressure in adjacent link BC. This pressure is equal to $w + 3.4641 V/L$. By equating these two pressures and solving for V, V is determined to be equal to $W_{sc} L/5.4641$, or for this case, $V = 834 \times 8/5.4641 = 1{,}222$ lb (5.4 kN).

For the condition shown in Fig. 10, it can be conservatively assumed that a shear equal to one-half of the concentrated load be applied at each side of joint B, provided $L > H$. However, distribution of the load on the pipe can be determined from Boussinesq's formula, as above, or from various approximations such as the one shown in Fig. 11. With this approximation for the case at hand, $qB_b = P$ and B_b in the denominator is considered equal to zero; the maximum loading on the pipe due to the surface load with an impact factor of 1.50 is $4 \times 10{,}000 \times 2.5^2 \times 1.5/(3 \times 3 + 2.5)^2 = 1{,}913$ lb/ft (27.9 kN/m). The loading is distributed along the link as shown in Fig. 12. The equivalent end shear, V, for this loading taking moments about C is $V = (1.5 \times 1{,}913 \times 7.25 + 0.75 \times 1{,}913 \times 6)/8 = 3{,}677$ lb (16.4 kN). Using a shear transfer coefficient of 0.2679, $V_c = 0.2679 \times 3{,}677 = 985$ lb (4.4 kN), which, when substituted, permits the contact pressures to be readily determined. If w were 1,500 lb/ft, P_B and P_c, from the surface loading alone, would be 2,739 lb/ft (40.0 kN/m) and 1,583 lb/ft (23.1 kN/m), respectively.

For cases where the resultant of the forces imposed on a link falls outside the middle third of the link length, the distribution of pressure must be adjusted.

One of the common situations where this occurs is with buried pipelines subjected to lateral thrusts resulting from unbalanced hydraulic pressure at fittings. For simplicity, the case of thrust at a tee, as shown in Fig. 13, will be considered herein.

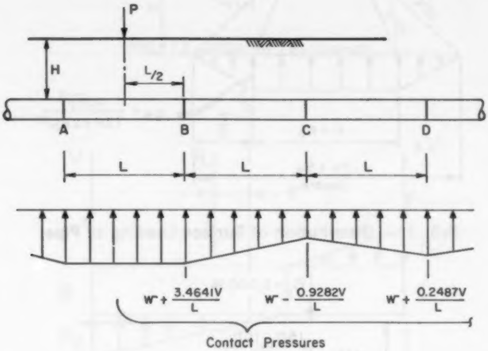

FIG. 9.—Concentrated Surface Loading Centered on Link

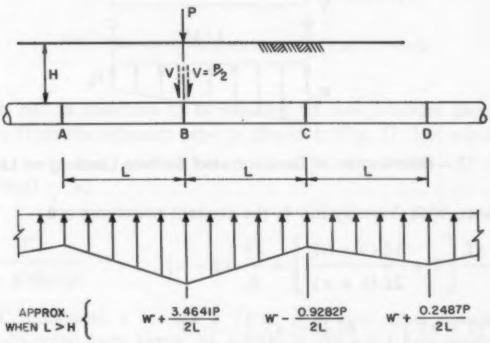

FIG. 10.—Concentrated Surface Loading over Joint

Basic Assumptions 1, 3, and 4 are retained for the following derivation. As before, a link BC is isolated, as shown in Fig. 14. The resultant, R, of these loads is

$$R = V(1 + x) \qquad (23)$$

and the eccentricity of the loads is

$$e = \frac{L(1 - x)}{2(1 + x)} \qquad (24)$$

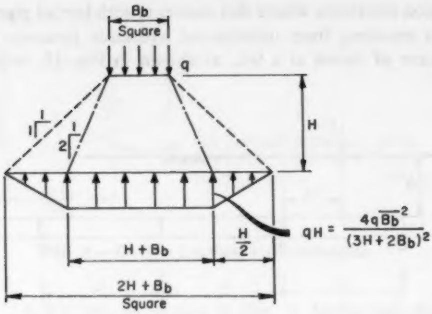

FIG. 11.—Distribution of Surface Loading to Pipe

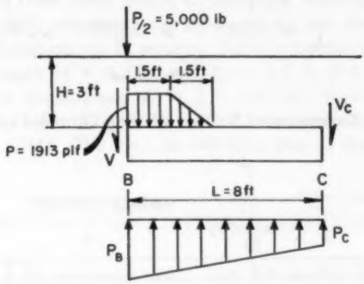

FIG. 12.—Distribution of Concentrated Surface Loading on Link

In accordance with Assumption 4, the contact pressures are

$$P_B = \frac{V(1+x)}{L}\left[1 + \frac{6L(1-x)}{2L(1+x)}\right] = \frac{2V}{L}(2-x) \quad \text{. . . . . (25)}$$

and $$P_c = \frac{V(1+x)}{L}\left[1 - \frac{6L(1-x)}{2L(1+x)}\right] = \frac{-2V}{L}(1-2x) \quad \text{. . . . . (26)}$$

For link CD the shear at joint C equals $-xV$. Substituting this value in Eq. 17, the contact pressure at C becomes

$$P_c = \frac{-2xV}{L}(2-x) \quad \text{. . . . . (27)}$$

By equating the contact pressure at each side of joint C as shown in Fig. 15, the equation to determine the shear transfer coefficient is $x^2 - 4x + 1 = 0$, for which $x = 0.2679$. This is equivalent to the case shown in Fig. 2.

The values of shear transfer coefficients at the other joints are powers of x. The pressures that result are shown in Fig. 16. The thrust, T, at a tee,

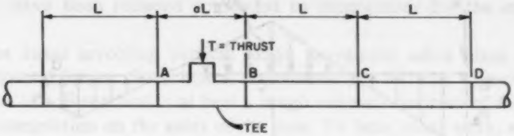

FIG. 13.—Plan of Tee Branch

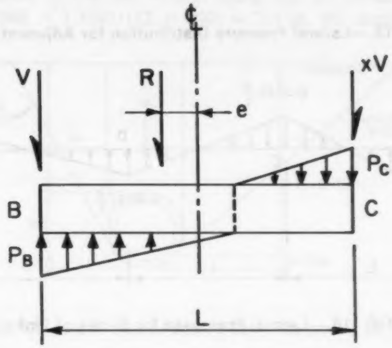

FIG. 14.—Link Subjected to Lateral Loading

with a length aL, is assumed to be resisted by soil pressure along its length, and the shear from the adjacent pipe as shown in Fig. 17. For equilibrium

$$T = 2V + 3.4641 \frac{V}{L} aL \qquad (28)$$

$$\text{or} \quad V = \frac{T}{2 + 3.4641a} \qquad (29)$$

Example C.—Assume a tee on a 27-in. (690-mm) diam concrete pressure pipe to be subjected to a thrust of 20,000 lb (89 kN), and (refer to Fig. 13) L = 16 ft (4.9 m), aL = 5 ft (1.5 m), and B_c = 3.25 ft (0.99 m). Determine the depth of cover required for stability, provided that

$$K_a = 0.34; \quad K_p = 3.00; \quad \gamma = 110 \text{ lb/ft}^3 \ (1{,}800 \text{ kg/m}^3) \qquad (30)$$

[The reader may refer to (6) for a detailed discussion of the factors which relate to the design for unbalanced thrust.]

Substituting, the value of the end shear and the maximum contact pressure become 6,392 lb (28.4 kN) and 1,384 lb/ft (20.2 kN/m), respectively.

Ignoring, for this example, the frictional resistance due to the vertical loading on the pipe and the weight of the pipe and its contents, the resisting force, with a safety factor of 1.0, is

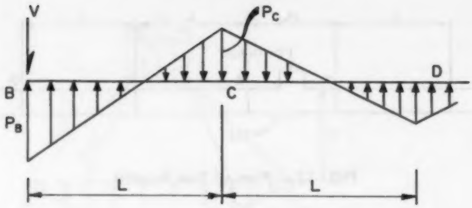

FIG. 15.—Lateral Pressure Distribution for Adjacent Links

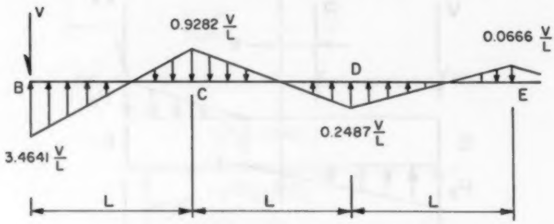

FIG. 16.—Lateral Pressures for Series of Links

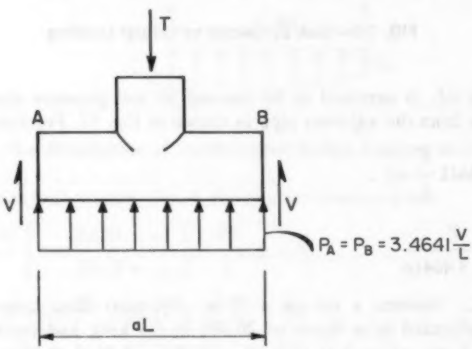

FIG. 17.—Plan of Tee

$$P_p - P_a = \frac{\gamma \times (H + 0.85B_c)}{2}(K_p - K_a);$$

substituting: $1{,}384 = \dfrac{110\,(H + 0.85 \times 3.25)}{2}(3.00 - 0.34);$

solving: $H = 6.7$ ft (2.0 m) . . . . (31)

It should be noted that the depths of the active and passive wedges, as shown

in Fig. 18, have been reduced somewhat to compensate for the curvature of the pipe.

As in the cases involving vertical loads, movement takes place as a result of the horizontal forces. Such lateral movements are difficult to ascertain since the modulus of soil reaction is at best a rough estimate because of the unreliable degree of compaction on the sides of the pipe. To date, most work, with respect to the soil modulus, has been in connection with flexible pipe (5). However, if for the previous example a modulus of 100 psi (700 kPa) were assumed, the lateral movement of the tee would, using Eq. 37 developed in Appendix I, be equal to (2.088 × 1,384)/(12 × 100) = 2.4 in. (61 mm). Such movement

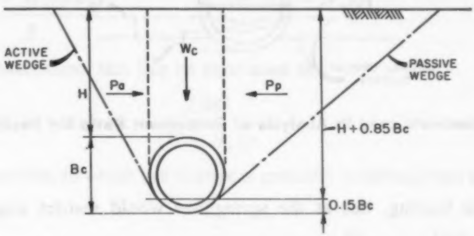

FIG. 18.—Active and Passive Soil Pressure Wedges

could open the pipe joints and constitute failure. Accordingly, a thrust block would most likely be required.

Summary and Conclusions

Settlement-modification and load transfer effects of flexible jointed buried pipelines can result in joint failure of such piping, the consequence of angular distortion between individual pipe sections (links). Having made certain assumptions concerning the pipeline (100% effectiveness of shear transfer; constancy of imposed loads; equal link lengths; longitudinal rigidity; and proportionality of load-related settlement to contact pressure), the behavior of buried piping can be calculated with some precision. Similarly, conditions contributing to joint failure and link displacement can be analyzed and preventive/corrective measures evaluated.

Appendix I.—Assumption Critique

Assumption 1 indicates that the joints are hinges and are fully efficient in transmitting shear. Modern gasketed joints provide little clearance between the bell ID and the spigot OD. Accordingly, the maximum relative movement that could take place would be equal to this clearance. The resistance due to gasket deformation of itself can develop little moment resistance, considering the tapered configuration of the joint and a gasket deformation of 15%–50%. Should ties other than at springlines be used, or pipe such as the locked mechanical joint type be used, it could restrict the pipe and cause bending at joints. In the

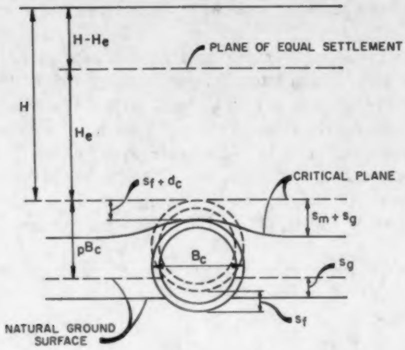

FIG. 19.—Settlements used in Analysis of Settlement Ratio for Positive Projecting Pipe

case of lateral loading, ties at the springlines would restrict angular opening of joints.

Assumption 2 indicates that the load on the pipe remains constant, unaffected by settlements caused by shear transfer. However, it has been stated that loading, at least for positive projecting conduit conditions, is settlement-dependent. With reference to notation shown in Fig. 19, the settlement ratio is defined as

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \qquad (32)$$

If rigid pipe is considered, d_c is zero, and this equation may be rewritten as

$$r_{sd} = 1 + \frac{s_g}{s_m} - \frac{s_f}{s_m} \qquad (33)$$

Also, $$s_m = \frac{H - H_e}{E_s} pB_c \qquad (34)$$

and $$s_g = \frac{H - H_e}{E_s} B_c \qquad (35)$$

s_f can be considered to be made up of two components—the general settlement equal to s_g and settlement, s, due to consolidation resulting from localized stress under the pipe. By adapting the general settlement equation for footings (7)

$$s = \frac{\Delta P B_b (1 - \mu^2) 3.40}{E_s} \qquad (36)$$

If $B_b = 0.7B_c$ and $\mu = 0.35$

$$s = \frac{2.088 \Delta P B_c}{E_s} \quad (37)$$

Substituting in Eq. 33

$$r_{sd} = 1 + \frac{s_g}{s_m} - \frac{s_g}{s_m} - \frac{s}{s_m}; \quad \text{or} \quad r_{sd} = 1 - \frac{s}{s_m};$$

$$r_{sd} = 1 - \frac{\dfrac{2.088 \Delta P B_c}{E_s}}{\dfrac{(H - H_e) p B_c}{E_s}}; \quad r_{sd} = 1 - 2.088 \frac{\Delta P}{(H - H_e) p} \quad (38)$$

For a given installation, this can be expressed as

$$r_{sd} = 1 - k \Delta P \quad \text{in which} \quad k = \frac{2.088}{(H - H_e) p} \quad (39)$$

With shear transfer, in which the change in pressure resulting from shear transfer $= \Delta P_V$

$$(r_{sd} + \Delta r_{sd}) = 1 - k(\Delta P + \Delta P_v); \quad \Delta r_{sd} = -k \Delta P_v; \quad \frac{\Delta r_{sd}}{r_{sd}} = \frac{-k \Delta P_v}{1 - k \Delta P}; \quad (40)$$

If $\Delta P_v / \Delta P = 1.0$, then

$$\frac{\Delta r_{sd}}{r_{sd}} = \frac{-k \Delta P}{1 - k \Delta P} \quad (41)$$

and if $r_{sd} = 0.7$, $k \Delta P = 0.3$

$$\left.\begin{array}{l} \text{then} \quad \Delta r_{sd} = 0.7 \dfrac{-0.3}{1 - 0.3} = -0.3 \\ \text{or} \quad r_{sd} + \Delta r_{sd} = 0.7 - 0.3 = 0.4 \end{array}\right\} \quad (42)$$

If $p = 0.7$

$$\left.\begin{array}{ll} \underline{\Delta P \text{ only}} & \underline{\text{with } \Delta P_v \text{ added}} \\ r_{sd} p = 0.7 \times 0.7 \simeq 0.5 & r_{sd} p = 0.70 \times 0.4 \simeq 0.30 \\ C_c \simeq 1.5 \dfrac{H}{B_c} & C_c \simeq 1.4 \dfrac{H}{B_c} \end{array}\right\} \quad (43)$$

Since $w_c = C_c \gamma B_c^2$, the ratio of these loads is

$$\frac{1.5 \dfrac{H}{B_c}}{1.4 \dfrac{H}{B_c}} = 1.07 \quad (44)$$

Accordingly, based on the foregoing, there would be an equalizing effect on the contact pressures due to change in imposed loads. Since loading and contact pressures are interdependent and self-adjusting, it is reasonable to assume that the maximum change in loading that would actually be realized would be about 3% or 4%. Therefore, for all practical calculations, Assumption 2 would be valid.

Assumption 3 requires that the link lengths are equal, other than that of the link adjacent to a structure. It is normal construction practice to use the maximum standard section length of pipe that can be economically placed. Shorts are used where required at structures for closure, or where alignment is changed with joint deflections. Therefore, except for these special cases, the assumption is generally valid.

Assumption 4 requires that the links are rigid longitudinally. If a link is considered as a beam on an elastic foundation, the rigidity criterion is

$$\lambda L = \sqrt[4]{\frac{KL^4}{4EI}} \qquad (45)$$

Based on the proposals of Hetenyi (4) and others, the pipe would be rigid longitudinally, provided that λL is less than 0.8. Most pipe meets this criterion. For example, consider an 18-in. (460-mm) diameter ductile iron pipe with a length of 18 ft (5.5 m). After deducting the casting tolerance and the service allowance for a class 50 pipe, the net wall thickness becomes 0.20 in. (5.08 mm). The resulting moment of inertia for the pipe is equal to 540 in.^{4} (2.25 dm^4). Using $E = 24 \times 10^6$ lb/in.^{2} (165 GPa), a bearing width equal to half the pipe outside diameter, and a K_s value of 200 lb/in.^{3} (54.3 MPa/m) (which is representative of a medium to dense sand), the λL value is about 0.25, well within the 0.8 criterion. This criterion would not be exceeded unless the K_s value increased a hundredfold.

Assumption 5 presumes that settlements are proportional to the contact pressures. Provided that the deformations are small, settlement of the pipe is reasonably proportional to the increase in loading. When the loading approaches the ultimate value, the settlement is disproportionate to loading—caused by an almost plastic flow and displacement of soil from under the pipe.

Appendix. II.—References

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Appendix III.—Notation

The following symbols are used in this paper:

A, B, C = pipe joint designations;
a = ratio of length of link adjacent to structure to others;
B_b = contact surface width between pipe and subgrade, in feet (meters);
B_c = outside width of pipe, in feet (meters);
C_c = load coefficient for use in Marston's formula for earth loads on rigid and flexible positive projecting conduits;
C_s = surface load coefficient;
d_c = deflection of the pipe, i.e., shortening of its vertical dimension, in feet (meters);
E = modulus of elasticity of the pipe material, in pounds per square inch (Pascals per square meter);
E_s = modulus of elasticity of soil, in pounds per square foot (Pascals per square meter);
e = eccentricity of resultant, in feet (meters);
F = surface load impact factor;
H = earth cover over pipe, in feet (meters);
H_e = height of column of soil from plane of equal settlement to critical plane, in feet (meters);
I = moment of inertia of the pipe cross-section, in in.^4 (cm^4);
K = $K_s \times$ (width of contact surface), in pounds per square foot (Pascals per square meter);
K_a = coefficient of active soil pressure;
K_p = coefficient of passive soil resistance;
K_s = modulus of subgrade reaction, in pounds per cubic inch (Pascals per cubic meter);
$k = 2.088/[(H - H_e)p]$ = constant for given pipe installation;
L = length of link, in feet (meters);
$N = (N_c w_c + w_p + w_w)/w$ = ratio of loading at joint A to that of joint B;
N_c = ratio of earth loading for complete projection condition to that of incomplete project (Fig. 7);
P = concentrated surface load, in pounds (newtons);
P_A, P_B, P_C = contact soil pressure under pipe at joints A, B, C;
P_a = total lateral active soil pressure, in pounds (newtons);
P_p = total lateral passive soil resistance, in pounds (newtons);
p = projection ratio;
R = resultant of loads on link, in pounds (newtons);
r_{sd} = settlement ratio;

S_B, S_C = pipe settlement at joints B and C, in feet (meters);
S_m = non-load-related settlement, e.g., bedding adjustment, in feet (meters);
S_w = settlement of pipe due to soil compressibility, in feet (meters);
s = pipe settlement, in feet (meters);
s_g = settlement of natural ground adjacent to the pipe, in feet (meters);
s_m = compression of the column of soil of height pB_c, in feet (meters);
s_f = settlement of the bottom of the pipe, in feet (meters);
T = thrust at fitting due to internal pressure, in pounds (newtons);
V = joint shear, in pounds (newtons);
W_{sc} = load on pipe due to concentrated surface load P, in pounds (newtons);
$w(w_c + w_p + w_w)$ = uniform loading on link, in pounds per foot (newtons per meter);
w_c = uniform earth loading per unit length on pipe for incomplete projection, in pounds per foot (newtons per meter);
w_p = weight of pipe per unit length, in pounds per foot (newtons per meter);
w_w = weight of pipe contents per unit length, in pounds per foot (newtons per meter);
x = shear transfer coefficient;
$\alpha_A, \alpha_B, \alpha_C$ = angular deflection of links AB, BC, CD;
γ = unit weight of soil, in pounds per cubic foot (kilograms per cubic meter);
ΔP = change in contact pressure, in pounds per square foot (Pascals per square meter);
ΔP_y = change in contact pressure due to shear transfer, in pounds per square foot (Pascals per square meter);
$\theta_A, \theta_B, \theta_C$ = angular joint openings at joints A, B, C, in degrees (radians);
λL = rigidity criteria; and
μ = Poisson's ratio for soil (assumed as 0.35).

To avoid expressions that might appear to be too cumbersome, the author has not used the radical forms for the various numerical values. For those that prefer the purer form, the following equivalents may be used; $0.0192 = 26 - 15\sqrt{3}$; $0.0666 = 52\sqrt{3} - 90$; $0.0718 = 7 - 4\sqrt{3}$; $0.2487 = 14\sqrt{3} - 24$; $0.2679 = 2 - \sqrt{3}$; $0.9282 = 4\sqrt{3} - 6$; $1.7321 = \sqrt{3}$; $3.4641 = 2\sqrt{3}$; $5.4641 = 2 + 2\sqrt{3}$; $4.3923 = 6\sqrt{3} - 6$; and $6.9282 = 4\sqrt{3}$.

PIPELINE CORRIDORS IN DEVELOPING AREAS[a]

By Buckley L. Ogden,[1] M. ASCE

(Reviewed by the Pipeline Division)

INTRODUCTION

Special purpose agencies are formed to provide a specific service to the public which may permit an area to develop. The fact that development can occur will, in many instances, subject the agency to pressure from this development.

This has been the case with the right-of-way utilized for water transmission facilities owned and operated by the San Diego County Water Authority (SOCWA).

The purpose of this paper is to present some of the problems that have been experienced by this agency and, more importantly, bring the problem of right-of-way encroachment to the attention of engineers and managers as a very real and important problem.

BACKGROUND

The SOCWA is a public corporation organized under the County Water Authority Act approved by the Legislature of the State of California on May 17, 1943. Formal organization of the SOCWA was completed on June 9, 1944.

The County Water Authority Act provides for the organization, incorporation, and government of county water authorities, and authorizes and empowers such authorities to acquire water and water rights. The act also provides for the acquisition, construction, operation, and management of works and property by these authorities, and provides for the incurring of bonded indebtedness therefore as well as for the taxation of property within such a county water authority.

The SDCWA has the right of eminent domain, including the condemnation of private property for public use, similar in rights, powers, and privileges to those of a municipal corporation.

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The rights to acquire water and water rights may be exercised within the State of California or outside the State; however water and water rights may not be acquired within the county in which the SDCWA is located. Water imported to the SDCWA service area can be transported and stored but must be sold at wholesale to public agencies that are a part of the SDCWA. At this time, the SDCWA has 24 member agencies that may receive water.

The SDCWA is a member agency of The Metropolitan Water District of Southern California (MWD), together with 14 cities and 12 municipal water districts, making a total of 27 member agencies. The MWD has entitlements to Colorado River water and contracts with the Department of Water Resources for State Project water from Northern California for delivery to these 27 member agencies.

The SDCWA receives all of its water from the MWD at a wholesale cost, and in turn, applies a surcharge to cover operation and maintenance costs, plus a recently-added charge for capital outlay costs. All water is delivered to the SDCWA member agencies at wholesale.

The importance of the SDCWA activities may best be demonstrated by the amount of water importedto the San Diego region, which in recent years has been 80%–90% of the total water used in its service area. This total water use has been 400,000 acre-ft/yr–460,000 acre-ft/yr (493,000 dekameters^3/yr–567,000 dekameters^3/yr).

During World War II, the United States Government, particularly the United States Navy, increased its activities in San Diego County, and in November 1944, with the use of water growing rapidly during a period of drought, President Roosevelt appointed a committee on water problems in San Diego and, subsequently, directed immediate construction of the San Jacinto-San Vicente Aqueduct. Because of a drought, time was of the essence to insure that capacity from this initial pipeline would be available when required. However, the SDCWA was not yet staffed for a major design and construction project such as this and, in addition, was not yet a member of the MWD. For these reasons, the Bureau of Reclamation, now known as the Water and Power Resources Service, was tasked with the design of this facility. The United States Navy Bureau of Yards and Docks, now the Naval Facilities Engineering Command, administered the construction contract.

Annexation to the MWD was completed in April 1946, some 2 yr after design was authorized.

At the time the first pipeline was being planned, it was recognized that a second pipeline would be required almost immediately following construction of the initial pipeline. Therefore, tunnels and other common facilities were sized to accommodate the capacity of two pipelines. The rights-of-way for this first aqueduct were acquired in the name of the United States.

On November 26, 1947, water began flowing into the San Vicente Reservoir through Pipeline No. 1 under operational control of the SDCWA.

Subsequent Development

San Diego County has grown considerably since 1952, the year the second pipeline was completed. Since completion of the second pipeline, which has a capacity of 100 cu ft/sec (2.8 m^3/s), identical to the first pipeline, the SDCWA

has constructed two additional pipelines to supply the needs of its member agencies by doubling the existing capacity in 1960 with the construction of a third pipeline having a capacity of 250 cu ft/sec (7.0 m^3/s), and again doubling the capacity by constructing a fourth pipeline in 1971 with a capacity of 450 cu ft/sec (12.6 m^3/s). At the present time, an additional pipeline having a capacity of 500 cu ft/sec (14.0 m^3/s) is under contract that will meet projected demands to the year 2000.

Problems Encountered

At the time the first and second pipelines were under construction, much of the alinement was in open, undeveloped country. In many cases, because the aqueduct was located in very remote locations, the ground surface was not restored, the pipelines were placed relatively shallow, and little concern was given to future utilities and roads that might cross the pipelines without costly remedial facilities to insure the structural integrity of the pipelines.

By contract agreement, the SDCWA is purchasing the two pipelines from the United States for that portion of the aqueduct under its operations and maintenance control; however the rights-of-way remain in the name of the United States.

The fact that the SDCWA operates and maintains pipelines and other facilities within the right-of-way under the control of the United States Government complicates the administration of encroachments, crossings, and other developments by landowners adjacent to this portion of the SDCWA aqueducts by requiring permits from both governmental agencies.

During construction of the third pipeline, land use patterns were starting to develop in the county, and the pipeline was designed, in part, to compensate for this future development as visualized at that time. Unfortunately, the county had not yet adopted a county-wide development plan, and the extent of this development was sadly underestimated. Rights-of-way, in many cases, were extremely narrow; in many places they were only 20 ft in total width in steep longitudinal and side-hill reaches.

Agricultural use of the rights-of-way for annual crops was, in most cases, permitted under provisions contained in SDCWA easement documents. In many cases, permanent crops have also been permitted on a case-by-case basis provided access roads were not disturbed.

The influx of people to the San Diego region has caused development in many of the previously remote areas, and the increasing value of land causes subdividers to crowd as close to the SDCWA easements as possible. In many cases, cut or fill banks are coincident with the right-of-way lines making maintenance of the 4-ft–7-ft (1.2-m–2.1-m) inside diam pipelines very difficult, and the removal and replacement of a section of pipe nearly impossible if confined to the right-of-way. Fig. 1 shows the proximity of development to the pipeline right-of-way. The width of the right-of-way at this location is 20 ft (6.1 m). Irrigation of vegetation on slope banks causes the right-of-way to be muddy constantly. Damaged gates and other acts of vandalism are a constant problem in a developed, as well as an undeveloped area. It is questionable whether repairs requiring excavation of the pipeline could be contained within the right-of-way; see Fig. 2 for the schematic cross section of this condition.

During the design phase of the fourth pipeline, it was fairly well-recognized that development would continue and the pipeline should be designed to support highway loads and deep enough to permit sewers, water distribution, and other

FIG. 1.—Highway-Type Cut Section

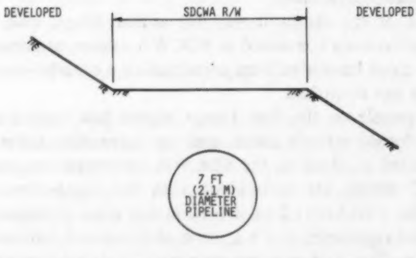

FIG. 2.—Typical Section with Slope Banks Starting at Right-of-Way Lines; Buildings for Home or Business Can Also Abut Easement with No Setback Requirement

utilities to pass over the line with reasonable clearance, and that the ground surface should be restored to its original elevation unless property was acquired with fee title. With the third and fourth pipelines being in a common easement,

this is somewhat contradictory in that the profile blocked is, in many cases, nearly twice the pipe diameter as indicated in Fig. 3.

Rationalizing this situation, when development occurs only one pipeline will be affected and corrective action will involve one-half, or less, of the SDCWA capacity. This is becoming more important each year as the annual use of this capacity increases. Many recent encroachments over the rights-of-way have indicated that this choice was correct.

In addition, the SDCWA attempts to coordinate its planning with other agencies to insure that rights-of-way can be shared with other facilities, particularly roads and urban streets.

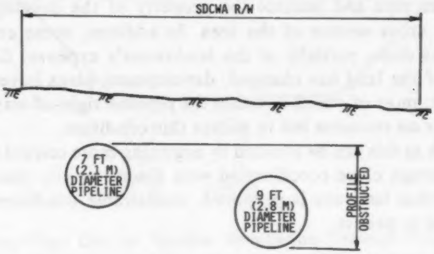

FIG. 3.—Pipelines at Different Depths Obstruct Lateral Profile Somewhat Higher Than Either Pipe Diameter

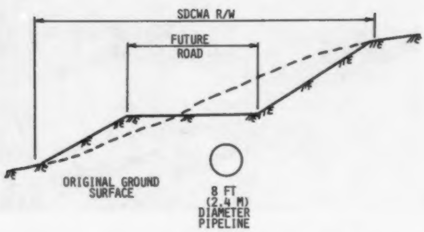

FIG. 4.—Construction of Aqueduct Included Grading for Approximately 4,000 ft (1,220 m) of Future Roadway

A prime example of this type of common use is the cooperation between the SDCWA and the city of San Diego for a pipeline recently completed. This is in an area of northeast San Diego where a major collector street, Jackson Drive, has been planned, and the SDCWA acquired its right-of-way coincident with the street alinement. The installation of the pipeline required considerable grading, but this would be required for the road with the ultimate result being that less property is encumbered by rights-of-way for public utilities.

A major advantage to the SDCWA, in this particular case, is that there will be no development adjacent to the right-of-way that could cause drainage, erosion, or other hazards to the aqueduct not anticipated in the pipe design. This installation is shown in Fig. 4.

A situation which does not have a happy ending for the SDCWA occurred in 1970 with a private developer. During negotiations for additional rights-of-way for the fourth pipeline, the developer's tentative plan indicated that a regional shopping center would be constructed on a large parcel straddling the aqueduct easement. The SDCWA was asked to place its pipelines at an elevation some 50 ft (15.3 m) below the natural ground line so a relatively flat area could be generated for the shopping center. This required a deep highway-type cut, a section of which is shown in Fig. 5; in this section a 69-in. (1,750-mm) prestressed concrete cylinder pipe was relocated and a 96-in. (2,440-mm) prestressed concrete cylinder pipe was installed. Pipe sections in the background of Fig. 5(*a*) and in the left center of Fig. 5(*b*) are removed sections of 69-in. (1,750-mm) prestressed concrete cylinder pipe and became the property of the developer; see Fig. 6 for a schematic cross section of the area. In addition, some embankment for the pipelines was done, partially at the landowner's expense. Since that time, the ownership of the land has changed, development plans have also changed, and this deep cut, most of which is within the pipeline right-of-way, is to remain. The SDCWA has no recourse but to accept this condition.

Situations such as this can be avoided by acquiring more control of the adjacent property and through close coordination with county or city planning to insure that no matter what land use is proposed, undesirable conditions such as this are not permitted to persist.

(*a*)

(*b*)

FIG. 5.—Highway-Type Cut Section for Proposed Regional Shopping Center

Mention has been made of subdivisions with boundaries coincident with the right-of-way boundary. Recent heavy storms in the Poway area of San Diego County made this problem surface dramatically for two homeowners.

The aqueduct right-of-way crosses a hillside perpendicular to the flow of runoff and crosses several not too evident drainage depressions. A subdivision consisting of possibly 40 lots was planned, nine of the lots abutting the downhill right-of-way line with an 8-ft–10-ft (2.4-m–3.1-m) high cut slope topped with a brow ditch to intercept the runoff and divert it to drains designed to dispose of the runoff into natural channels. A typical section is depicted in Fig. 7.

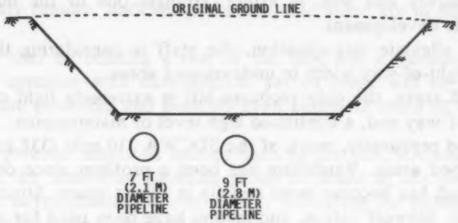

FIG. 6.—Highway-Type Cut for Pipeline Installation Through Proposed Area-Wide Shopping Center; Development Adjacent to Right-of-Way is Now Residential

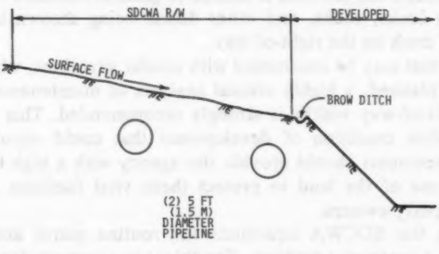

FIG. 7.—Development Adjacent to Right-of-Way Included Brow Ditch of Insufficient Cross Section and Lateral Slope to Provide Drainage

Unusually heavy rains caused the drainage depressions to fill, erode, and deposit sediment in the brow ditch. The result was severe erosion to the cut slope and deposition of about 4 ft (1.2 m) of sediment in and around two of the homes.

The major portion of the eroded area occurred over the aqueducts causing several thousands of dollars in damages which would not have occurred had this subdivision not been built in this location or if it had been more compatible with the natural terrain.

Unfortunately, the SDCWA has no jurisdiction outside the right-of-way boundaries and must rely on San Diego County planners to foresee problems of this nature.

An agency owning and operating its facilities in situations such as this is placed in a rather difficult position with some responsibility for a situation created by others. However, it is the homeowner who suffers the greatest loss.

As development continues in the County adjacent to the SDCWA rights-of-way, problems similar to these will become more frequent and consume more effort by the staff to insure that pipelines are not jeopardized to the point that service is disrupted.

The SDCWA is becoming more cognizant of the potential problems in many cases because of past experiences. Without control of sufficient lands adjacent to the aqueducts, development will continue to occur much too close to the pipelines for safety and will, in effect, strangle one of the most necessary services for this development.

To partially alleviate this situation, the staff is considering the acquisition of additional right-of-way width in undeveloped areas.

In developed areas, the only recourse left is extremely tight control on the existing right-of-way and, a continued high level of maintenance.

As mentioned previously, much of the SDCWA 210 mile (338 km) of pipeline is in undeveloped areas. Vandalism has been a problem since construction of the pipelines and has become more serious in recent years. Structures housing air-relief valves, blowoff valves, and meters have been used for target practice with high-powered rifles and are subject to general nuisance vandalism. The remote right-of-way with fairly good maintenance roads makes these structures easily accessible, particularly to motorcycle and four-wheel drive vehicles. As development occurs, the problem is limited to general nuisance-type vandalism, such as broken locks, rocks, and other debris being thrown into structures, and disposal of trash on the right-of-way.

For agencies that may be confronted with similar situations, whether pipelines are existing or planned, a highly critical analysis of maintenance requirements in terms of right-of-way widths is strongly recommended. This should include the worst possible condition of development that could occur. In addition, right-of-way agreements should provide the agency with a high level of control in the surface use of the land to protect these vital facilities and the safety of adjacent property owners.

Access along the SDCWA aqueducts for routine patrol and maintenance functions is also a continuing problem. Conditions in easement documents contain provision for access and egress along the easement, but at the same time permit the surface use of the right-of-way so long as it will "avoid unreasonable interference" with the SDCWA use of the easement. A fence with adequate gates is probably not "unreasonable interference" but this can lead to extensive, additional costs in maintenance of the aqueducts where it is conceivable that gates could be located at each line in a subdivision.

In an effort to minimize this particular problem, each subdivision is asked to grant to the SDCWA an unobstructed access road right-of-way along the easement (in most cases within the pipeline easement) from dedicated roads through the limits of the parcel being subdivided.

This practice has been instrumental in the elimination of many gates and the "unreasonable interference" with SDCWA use of the easement in the form of lawns, gardens, and other minor uses. It should be mentioned at this point that even though the Authority has the "right of ingress and egress," public

relations suffer dramatically when vehicles or equipment are driven through a lawn or garden.

Summary

The purpose of this paper is not to suggest all encompassing solutions to right-of-way problems, but rather to indicate a few of the problems experienced by one agency with a rather short history of only 34 yr.

Some of the history is unique in that rights-of-way containing SDCWA-owned facilities is in the name of the United States Government; some rights-of-way are in extremely remote, undeveloped areas, and some are in highly-developed areas.

It is hoped that by relating a few of the SDCWA experiences, both good and adverse, agencies with development yet to come will be better able to cope with future problems and possibly even eliminate some of these potential problems.

One of the greatest assets an agency such as the SDCWA can have is good relations with the area-wide planning group to assist in the protection of its facilities from development and conversely, protecting the public from hazards that could be created by improper planning.

Personnel administering the operations, maintenance, and public relations in future generations will be highly appreciative of your foresight in recognizing the type of problems enumerated herein.

[illegible]

Rehabilitation of Sanitary Sewer Pipelines[a]

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(Reviewed by the Pipeline Division)

Introduction

The analysis and evaluation of sewer pipeline deterioration have led to a cost-effectiveness consideration of in-place rehabilitation, in lieu of total replacement. This paper presents *initial* findings on existing methods and materials utilized in the current state-of-the-art of sewer pipeline rehabilitation. A description of the techniques, new and existing, are treated in a comprehensive manner. A review of the various types of sewer pipeline failures, and problems related to overall deterioration, are described.

Consideration has been given to exterior pipeline rehabilitation, in addition to basic internal rehabilitation, with an emphasis on several grouting methods and materials for total rehabilitation.

Further details and subsequent conclusions will be covered in work ongoing in the ASCE Pipeline Planning Committee's Task Committee on "Sewer Pipeline Rehabilitation."

Evaluation of Problems

The elimination of "excessive" infiltration of extraneous waters through pipeline rehabilitation has been proven to be feasible on a cost-benefit ratio primarily due to the high cost of new replacement construction. Excessive extraneous flows into treatment facilities makes the cost of treatment an increased economic burden. In addition, the "infiltration," "exfiltration" of sewage from defective pipes creates a potential health hazard with the possible contamination of underground aquifers used for potable water supply.

Rehabilitation of deteriorated sewer pipelines is justified when the total evaluation of all existing data of the investigative and analysis process has been completed. Sewer system evaluation studies (SSES) are normally required

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Note.—Discussion open until December 1, 1981. To extend the closing date one month, a written request must be filed with the Manager of Technical and Professional Publications, ASCE. Manuscript was submitted for review for possible publication on November 5, 1979. This paper is part of the Transportation Engineering Journal of ASCE, Proceedings of the American Society of Civil Engineers, ©ASCE, Vol. 107, No. TE4, July, 1981. ISSN 0569-7891/81/0004-0497/$01.00.

by the Environmental Protection Agency (EPA) where federal funding is involved.

Sewers may be classified according to their design configuration such as circular, basket-handle, catenary, egg-shaped, horseshoe, ovoid, rectangular, semielliptical, or square. Obviously, the circular and elliptical shapes have been best suited for pipes. Most other shapes have been constructed in place. Materials of sewer conduit construction exist as reinforced concrete, vitrified clay, brick and clay tile, asbestos cement, plastics, reinforced plastics, steel, cast iron, and ductile iron. Not all configurations used in sewer conduit construction can be successfully rehabilitated.

The effects of ground water play an important part in the analysis of sewer pipeline rehabilitation. Ground water can, at times, rise and fall rapidly creating a soil pumping action adjacent to failing pipe. This has a pronounced effect on infiltration rates. Chemicals in ground water may have a detrimental effect on certain pipe and joint materials, and may accelerate induced electrolytic corrosion of metal pipes by stray currents. The movement of ground water can affect the foundation of pipes where backfill materials are not similar and compaction levels are inconsistent.

Sewer pipelines continue to fail in a variety of ways, and this continuance has become a potential hazard to the public. Several distinct features are quite apparent in failing sewers. The most common failures are either structural or corrosive in nature. These basic failures lead to other problems in sewers such as root intrusion, infiltration-inflow, and exfiltration. These occur primarily in gravity flow pipes. Corrosive failures are usually attributed to acid corrosion of pipe and joint materials susceptible to attack in the system. Structural failures are those attributable to sewer pipes, manholes, and appurtenant structures, where corrosion, poor construction, and improper design are evident.

The most apparent failure to most sanitary sewer pipelines is corrosion. More problems can be attributed to corrosion than any other type of failure. Corrosion failures are commonly found in the crown or invert of sewer pipes, joints where cement mortar was used, and other areas where acidic corrosion of various types and concentrations corrode, over time, the materials used. Corrosion of pipe inverts is usually caused by disposal of corrosive and erosive industrial wastes, not previously identified. External corrosion of some pipe materials is also evident in soils containing excessive pH ranges and, as previously stated, aggressive ground water.

Typical structural failures in sewer pipes are most commonly found in lines carrying raw sewage. These failures consist of broken bells and spigots, crown cracks, beam breaks, slab outs, and shear breaks. One of the most evident and continuing failure sources, structurally, is that of cement mortar joint deterioration found in concrete and vitrified clay pipes. Root intrusion of the deteriorated joints in most cases will start a structural failure of cracking and eventually breaking the bell or spigot, or both. A failure at the pipe joint will lead to a series of structural failures depending on local conditions. Exfiltration of sewage effluent through braken pipe joints can cause washouts and voids in the soil surrounding the pipe as well as increased loading on the pipe.

Infiltration of ground water can also create adjacent soil voids. Fine sands and other materials can enter the failing pipe through cracks at defective joints. The loss of the pipe zone backfill and bedding could cause settling and eventual street collapse. This occurrence is particulary noticeable during surcharging under

wet weather conditions. Soil voids are also responsible for beam breaks of small pipes. Shear breaks occur with excessive shifting or differential settlement of soil adjacent to the pipe. Shear breaks are also found in pipes that do not have a flexible connection adjacent to structures or manholes. Earthquakes are also responsible for shear breaks. Slab outs, usually found in lined and unlined concrete pipes, are small areas of the pipe wall that fall out, where very fast and highly acidic corrosive concentrations exist.

Crown cracks, found in concrete, clay, and asbestos cement pipes, are usually due to increased loading on a pipe in excess of the original design, improper pipe strength selection, and inadequate placement of foundation, bedding, and backfill materials. Sagging or bellies in sewer pipes are caused by poor compaction or displacement of bedding materials in localized areas.

Rehabilitation Methods and Materials

The best rehabilitation method chosen for any sewer pipeline renewal is the one that meets the parameters for solving the entire problem, which includes external as well as internal conditions. Considerations will include, but not be limited to:

1. Structural condition of existing pipeline.
2. Corrosive condition of existing pipeline.
3. Extent of internal debris (cleaning).
4. Extent of washouts and voids (external).
5. Evaluation of seismic potential and soil shifting, or settlement conditions related to pipe location (e.g., fault and swamp areas).

Several external conditions in form of minor cracks and joint openings can be renewed from the inside (to be described further herein); however, most external remedies are performed from above ground or by excavation adjacent to the problem pipe. The two common methods of solving problems of significant ground-water movement, washouts, settlement, and soil voids are *chemical grouting* and *cement grouting*.

External Rehabilitation

Chemical Grouting.—Chemical grouts consist of solutions of two or more chemicals that react to form a gel or a solid precipitate as opposed to cement or clay grouts that consist of suspensions of solid particles in a fluid. Reaction in the solution may be either chemical or physiochemical and may involve only the constituents of the solution with other substances encountered in the use of grout. The reaction causes a decrease in fluidity and a tendency to solidify and form occlusions in channels or fill voids in the material into which the grout has been injected.

Most applications of chemical grouts are made into saturated or partially saturated formations. The majority of chemical grouting applications are related to water shutoff rather than strength increase. Such is the case for sewer rehabilitation purposes.

The basic method of external chemical grouting is to place a relatively impermeable barrier called a *grout curtain* adjacent to or surrounding the pipe.

It consists of one or more interlocking rows of grouted soil cylinders. Each of the individual cylinders is formed by the injection of grout through a pipe or drilled hole which has been placed in the formation, adjacent to the pipeline location. These cylinders should have relatively uniform cross sections throughout their depth. For uniformity, it is important to grout short sections or stages of any individual hole so as to minimize the opportunity for grout to flow preferentially throughout the hole depth. In determining actual gel times, it will be found that these are directly related to pumping rates and to stage depth. All of these variables cannot be predetermined with great accuracy. The initial injections of the actual grout curtain are generally used to arrive at values for stage length, pumping pressure, and gel time. The diameter of the sewer pipeline, the size of problem area, the depth of sewer, and the ground-water zones help to determine the spacing between the grouting holes and the required number of rows. Economics usually limit the spacing between holes to 5 ft (1.5 m) or less and rows to 10 ft (3 m) or less.

One of the most widely used chemical grouts for curtain wall purposes, especially in fine soils, is the acrylamide type. Acrylamide grouts are mixed in proportions that produce stiff gels from dilute water solutions, when properly reacted. Several reactants and mixtures of reactants may be used. The gel is essentially impermeable to water. A beneficial property of the acrylamide chemical grout is that pumping times in excess of gel times can be utilized due to the low shear strength of the gelled grout. Gel times can be controlled by varying the temperature and the catalyst concentration. This property is effective in minimizing dilution with ground water and subsequent displacement of the grout. Chemical grouting with acrylamides in dry soils presents problems, and it is

TABLE 1.—List of Some Chemical Grouts and Sources of Supply

Type (1)	Trade Name (2)	Manufacturer, producer, or distributor (3)
Acrylamide	AM-9	American Cyanamid Co.
Acrylamide	PWG	Halliburton Oil Well Cementing Co.
Resin	Cyanaloc	American Cyanamid Co.
Resin	Herculox	Halliburton Oil Well Cementing Co.
Silicate	Injectrol-G	Halliburton Oil Well Cementing Co.
Silicate	Siroc	Raymond International, Inc.
Lignin	Blox-All	Halliburton Oil Well Cementing Co.
Lignosulfonate	Terra Firma	Concrete Chemicals Co.
Epoxy resin	—	George W. Whitesides Co.
Polyester resin	—	American Cyanamid Co.
Polyphenolic polymer	Terranier	Rayonier, Inc.
Resorcinol-formaldehyde	CR-726	Catalin Corp. of America
Phenoplast or resorcin-formol	—	Soletanche
Aluminum octoate	Firmgel	Byron Jackson, Inc.
Cationic organic-emulsion	SS-13	Brown Mud Co.

not recommended for external rehabilitation adjacent to sewer pipelines. Acrylamide is a toxic chemical that can be absorbed into the body through broken skin, inhalation, and swallowing. Because of this toxicity, potential hazards in handling and usage can occur if not supervised by technically qualified personnel. Additional chemical grouts are listed in Table 1.

Cement Grouting.—Portland cement grouts can be used for impermeabilization but are restricted in application to medium sands or coarser materials because of the larger size of cement particles. For purposes of filling voids and washouts adjacent to sewer pipelines, various portland cements are ideal. Unlike most chemical grouts, portland cement grouts are less expensive. Several types of portland cements have been used successfully. Type three (III) cement is often selected because of its fineness. A variety of water cement ratios can be used depending on subsurface conditions. Strength characteristics for void filling are not critical for soil surrounding buried sewer pipelines. In most cases, grout mixes yielding 500 psi (3,500 kPa)–1,000 psi (6,900 kPa), are adequate and easily obtainable with simple mix designs, attention to water-cement ratios, and injectionable mixtures. Clays can be added to cement for grouting to form gels, and to prevent settlement of the cement from suspension. They do, however, have the problem of no well-defined setting time, and they have a slow strength development. Because of this, they have not been utilized in sewer rehabilitation void grouting.

For extremely large void filling applications other cements such as pozzolan and fly ash mixes can be used and are more economical than straight portland cement and soil-cement mixtures. In addition to the above, portland cement grouts may be used as a filler and accelerator in silicate grouts and may be compatibly mixed with acrylamide to improve water shutoff capabilities and injectivity.

The following rehabilitation analyses will all be related to methods and materials used for internal sewer pipeline renewal.

Internal Rehabilitation

Chemical Grouting.—Chemical grouting is most commonly used to seal deteriorated and leaking pipe joints, service connections, and open joints. Small diameter pipes represent the largest area of renewal use; however, with special equipment, large pipes can also be grouted at the joints successfully. The two basic grouting materials used for internal grouting of sewer pipelines are acrylamide gel and polyurethane foam; however, other special mixes are available. Grouting with the acrylamide gel stops the leaks by internally injecting the grout to the soils surrounding the leaks to decrease their permeability. On the other hand, grouting with polyurethane foam seals the leaks by injecting the grout into the cracks and letting it solidify to form a barrier. Neither chemical can be used as a structural repair for broken, crushed, or cracked pipes.

A catalyst used in setting off and hardening of acrylamide gel has been found to be toxic and hazardous to workers. Polyurethane also reflects some degree of toxicity, but not as great as the acrylamide gel. At the present time, nontoxic gels, foams, and other materials are being developed and tested by U.S. and foreign manufacturers. It is anticipated that nontoxic materials will be available in the near future.

Before gelation, the acrylamide grouting mix has a viscosity very close to

that of water. This allows it to penetrate into small leaks and cracks in pipe walls and to mingle with outside soil particles. The acrylamide gel formed from the solution is a translucent, rubbery, and elastic material. Under moist conditions, the gel is resistant to attack by microorganisms, dilute acids, alkalies, and the ordinary salts and gases normally found in the ground. When the gel is formed in a soil matrix, the permeability of the soil is reduced. The degree of reduction of the permeability depends upon the extent to which the voids are filled with the gel. If they are completely filled, the gel-soil mixture is virtually impermeable.

When mixed with an equal amount of water, the polyurethane grout initially foams and then cures to a tough, flexible cellular rubber. The first stage of the reaction is referred to as the *foam time, induction time*, or *cream time*, and the second stage is called *cure time, set time*, or *get time*. Both the foam time and the cure time are temperature-dependent. Generally, the higher the temperature the shorter the reaction times. The grout is resistant to most organic solvents, mild acids, and alkalies and reflects a great degree of elongation with very little shrinkage.

Grouting of sewer pipe joints with acrylamide gel is generally accomplished with a sealing packer and a closed circuit television camera. The sealing packer is used to apply the chemical grout to the pipe joint. It is usually made of a hollow metal cylinder which has an inflatable rubber sleeve on each end of a center band. For the joint which needs sealing, the chemical grout is pumped into the space created between the two inflated sleeves. In this space, the grout and the initiator solutions are mixed together and squeezed out through the joint leak into the surrounding soil. There the grout displaces the ground water and fills the voids between the soil particles. A television camera is used to position, remotely, the packer on the pipe joint and to inspect the joints before and after the sealing operation. The sealing packer and television camera are pulled by cables through a sewer section from manhole to manhole. In addition, the air testing equipment is sometimes used to determine the integrity of the joints to check the effectiveness of the sealing.

For grouting sewers with the polyurethane foam, procedures similar to those with the acrylamide gel are followed. A sealing packer, similar to the one used for the acrylamide gel, is used for injecting the polyurethane foam. The packer is made of a hollow metal cylinder with three inflatable sleeves covered by a continuous outer sleeve. In operation, the packer is positioned on the joint to be grouted with the aid of television camera, and its end sleeves are inflated. The polymer and water are then introduced into the space created between the two inflated sleeves, and the foam time begins. At the end of the foam time, the center sleeve of the packer is inflated, forcing the grout into the pipe joint. After the cure time ends, the sleeves of the packer are deflated, and the packer is moved to the next joint.

To determine the integrity of the joints before and after the grouting, a water testing system, rather than an air testing system is usually used for the pressure testing.

In large diameter sewers where manual access is adequate, leaking joints of concrete and clay pipes can be injected with chemical grouts using a nozzle type applicator.

The initial low cost and speed of chemical grouting lends itself well to nonstructural sewer pipeline rehabilitation. Dewatering the line to be grouted,

undetermined ground-water movement, toxicity problems, and determination of set time have been deterrents to the use of chemical grouting.

Polyester Resin Lining (Cured in Place).—A polyester needle felt tubing, manufactured to the exact dimensions of the pipe to be relined, is saturated with a thermosetting resin. The purpose of this initially flexible tubing is to create a new pipe within the damaged original pipe.

This tubing is tailored to the exact diameter and length of the pipe to be renewed. It is inserted into the existing sewer from a manhole or other access points, or both, without the need for special excavations, with an attached angled inversion tube. Once the tube has been fully extended with cold water and pressure forcing it through the existing pipe and pressing the special felt material against the inner walls, the water is then heated, curing the resin saturated material and forming a hard impermeable pipe within the damaged original pipe.

The new lining is rather thin, 1/8 in. (3 mm) up to 3/4 in. (19 mm), and due to its smooth interior, flow capacities can be improved. As in most rehabilitation efforts, the existing sewer must be cleaned thoroughly to obtain the best results. This method allows for the filling of cracks, bridging gaps, articulation through pipe defects and small angles below 45°, depending on diameter. The resin tubing is not designed to carry total external earth loads as is the pipe. Inspection will reveal those structural areas where this method should not be used. Effective chemical resistance makes this process adequate for sewer corrosion parameters. Two chemically resistant resins are available for specific applications. A special cutting device is available to cut in-house service connections with the aid of a television camera to locate the laterial prior to cutting. Proper curing of the lining is important to insure successful attachment to the pipe wall at service connections so that no voids exist whereby sewage could penetrate behind the lining of the connection.

Direct savings of this method are in the form of no excavation, little traffic control, low labor intensity, and rapid installation. To date, the largest installation has been 30 in. (760 mm) in diameter. This method requires that the sewer be dewatered and cleaned prior to installation.

Comparatively, the polyester resin felt tubing method is economical; however, this will vary broadly with the diameter and length of the pipe to be renewed.

Sliplining or Pipe Insertion Renewal.—Insertion renewal, that is, inserting a new pipe material into the old deteriorated sewer pipeline has become an economical and popular method of rehabilitation. A variety of pipe materials may be used, and selection can be justified through certain features, benefits, and economics of materials and installation. Sliplining to date accomplished in the United States has been from 8 in. (200 mm)–108 in. (2,700 mm). At present, the following materials have been used in the insertion renewal process of rehabilitation:

Polyethylene Pipe (PE) (Extruded).—Polyethylene pipe is manufactured in three types; low, medium, and high density. The most commonly used for sewer sliplining is high density (HDPE). High density compounds are rigid and hard, strong, tough, corrosion-resistant, and well suited for sewer applications. Structural characteristics are a function of wall thickness design, as in most plastics and other homogeneous materials. Due to the low modulus of elasticity, careful design consideration should be evaluated in terms of buckling (ring buckling). Polyethylene pipes are categorized by the Standard Dimension Ratio (SDR),

and pipe grades are also classified by stress cracking resistance. Two national specifications are available for reference and design: they are ASTM D-1248 and ASTM D-3350. These standards apply generally to circular crosssection extruded PE pipe from 8 in. (200 mm–48 in. (1,200 mm).

Generally PE pipe is pulled through the existing sewer in long lengths. It is butt-fuse-welded above ground prior to pulling. The larger the diameter and thicker the pipe wall, the longer it takes to weld the joints together. The excavation required for this method is dependent upon the depth of the sewer, the diameter, and the wall thickness of the insertion pipe.

A large-diameter thick wall pipe with large entrance radii requires large excavation for installation purposes. The pipe is attached to a pulling head, and attached to a winch, usually electrical, at the other end of the sewer. Rollers are used above ground to facilitate the movement of the pipe during the pull. The pipe is pulled through the entire section at once. In some cases, the wall thickness design stresses cannot tolerate a single pull, and multiple pulls must be considered. Service connections can be connected by a remote connection system, heat fusion saddle, or tapping saddle. In most cases, except for some larger diameter sewers, service connections must be excavated, the connection made and then backfilled. Occasionally, when greater structural integrity is desired, the annular space may be cement grouted for the entire sewer length. This process also prevents ground-water migration and possible negative buoyancy (flotation) of the inserted pipe. Additional pipe design considerations should be investigated when grouting is utilized.

PE pipe insertion provides an excellent corrosion and abrasion-resistant sewer conduit with very good hydraulic characteristics due to its smooth inner surface. The flexibility of PE pipe allows it to articulate some offset in joints, manholes, and pipe changes. The fusion-welded joints, if properly fused and cured, are virtually leak free. In most cases, dewatering is required of the existing sewers; however, in some cases of low flow, the annular space may be adequate to convey the sewage.

Polyethylene Pipe (Spiral Welded).—This PE product is generally the same as the previous one with the following exceptions: the manufacturing process, size ranges, and design flexibility. Originally a German process, spiral welded PE pipe can be manufactured in sizes from 12 in. (300 mm)–144 in. (3,660 mm) in diameter, and in virtually any length depending upon the handling criteria. The load bearing strength of this pipe is a function of the wall thickness and homogeneous rib spacing. The rib spacing can be adjusted in thickness and length to meet the particular structural need of the pipe environment. This allows for excellent pipe economics, especially in large diameters. The joining of this type of high-density PE pipe is with a bell and spigot, sealed with an elastometric gasket or heat-fused joints. Physical and chemical properties are similar to circular extruded PE pipe. Due to the method of manufacturing, any diameter can be made to the specific dimensions required for the rehabilitation project. This allows for maximum diameter and a minimum annular space.

The installation of this product depends on the diameter and length specified. Economies are obtained by the excavation of a small installation pit when using bell and spigot pipe. In many instances, the pipe may be inserted while the sewer is flowing.

Polybutylene Pipe (PB) (Extruded).—Polybutylene (PB) resembles medium

density PE pipe in stiffness and chemical resistance, but has higher strength under sustained stress. It retains this strength with increasing temperatures. Its upper operating temperature is about 180° F (82° C) for gravity sewer applications. As a liner for the sewer renewal process, it is extremely good where extra protection is required for temperature and aggressive industrial waste effluents. Thinner walls may be used in design when compared to PE pipe. Standard specifications regarding PB pipe are ASTM D-2666 and ASTM D-3000.

The installation of PB pipe is virtually the same as PE pipe; however, the diameter ranges are currently limited to 24 in. (600 mm).

Reinforced Plastic Mortar Pipe (RPMP).—The use of RPMP for rehabilitation of sewers and pipelines has been significant. RPMP can be used to reline extensively cracked, corroded, and severly deteriorated existing sewers of all types. Essentially RPMP is inserted into the existing deteriorated sewer using 20 ft (6 m) lengths and a variety of installation methods. At present, diameters ranging from 18 in. (450 mm) to 108 in. (2,700 mm) are available from at least two manufacturers.

RPMP is a thermosetting composite of fiberglass, polyester, and other resins, combined with resin sand matrix. Commonly, each diameter has a specified minimum wall thickness based upon stiffness factors developed from ring compression, ring buckling, and longitudinal compression criteria. RPMP possesses very good chemical resistance and is corrosion-resistant for sewer applications. Hydraulic friction is low due to the very smooth interior surface, and the pipe has good abrasion and erosion resistance. High strength is apparent with RPMP as it exhibits a high modulus of elasticity when compared to thermoplastics.

One manufacturer has developed an inverted bell and spigot joint using an elastomeric gasket for sealing. The flow restriction with this joint is inconsequential, and it allows for a constant outside diameter, in addition to a large cross-sectional area to push or jack against. National specifications relating to design, materials, and testing of RPMP are ASTM D-3262 and ASTM D-3517.

The installation of RPMP for relining purposes requires an excavated pit approximately 30 ft (9 m) in length and 8 ft (2.4 m) or more in width. The existing sewer must be cleaned prior to the insertion, and restrictions and alignment parameters determined in order to properly size the liner. After the pit is dug, the existing sewer is exposed, and the top half of crown is removed in the pit area. Generally, RPMP can be inserted while the sewage is flowing. A half-full sewer for most diameters provides adequate flotation for lining distances up to 5,000 ft (1,500 m) from one installation pit. Lower flows reduce the insertion distances. RPM pipes are usually pulled or pushed upstream against the sewage flow. In many cases, the installation pit can be set in the center of the pipeline reach to be relined, whereby the pipe may be pulled or pushed from two directions. Pushing by jacking devices and pulling with cables and winches provides for rapid installation of RPMP. Additional installation pits may be required where angular offsets occur. Pipe joint offsets are small, and relining curvatures are limited to very small allowable angles.

Grouting the annular space between the pipes at the manhole connections is required, but grouting of the annular space for the entire pipeline is usually not required if the liner is strong enough to withstand the anticipated loads

in event of collapse. High ground water and excessive structural deterioration usually are the determinants for annular space grouting.

Minimum traffic obstruction, lightweight, high rate of installation, and reasonable cost are the apparent advantages of RPMP used for rehabilitation. Limitations in its use consist of the difficulty in making service connections and the minimum allowable curvature criteria.

Reinforced Thermosetting Resin Pipe (RTRP).—Similar in many respects to RPMP, RTRP has been used for rehabilitating sewers. It is inserted into the existing sewer usually by pulling it through. RTRP is manufactured in diameters from 8 in. (200 mm) to 192 in. (4,900 mm) and in lengths up to 80 ft (24 m) depending on diameter. A variety of joining systems are available and consist of bell and spigot with an elastomeric gasket, field-wrapped, and locking tension types. Either one may be used in sewer relining.

RTRP is manufactured from polyester resins, other corrosion-resistant resins, and fiberglass filaments wound on a mandrel or applied inside a mandrel as a composite. It exhibits high axial and longitudinal strength in addition to good abrasion and erosion resistance. A smooth interior provides low friction factors.

Installation excavations for RTRP are similar to RPMP, but can vary due to the choice of lengths and the joining method employed. The existing sewer must be cleaned. Dewatering is necessary when using RTRP, unless the annular space has enough capacity to accept the flow. If field-wrapped joints are used, they should be assembled above ground. Special pulling heads must be made for attachment to the pipe. The cable and winch system pulls the pipe from one end of sewer to the other usually from one installation pit. In some cases, pushing and pulling can be accomplished together. Using thin walls, varying lengths, and joining methods, RTRP has additional flexibility in negotiating slightly larger angular changes and pipe offsets in the existing sewer than RPMP.

Annular space grouting is virtually the same as that described for RPMP.

The advantages of using RTRP for sliplining are lightweight, high strength, minimum disruption of traffic, and a variety of joining systems. Basic disadvantages are high initial material cost, difficulty in making service connections, and the limitation of angular deflection.

Cement Mortar Lining (In Place).—Internal surface deterioration can be renewed in deteriorated sewer pipes made of concrete, steel, and iron by applying cement mortar on the interior surface. Lining material is usually a 1:2 portland cement mortar, using a high sulfate resistant cement. Lining thicknesses are usually 1/4 in. (6 mm) and up. Thickness depends upon the condition of existing pipe. In pipe diameters below 24 in. (610 mm), reinforcing is not normally required; however, in diameters above 24 in. (610 mm) that are badly deteriorated, it is desirable to reinforce the cement-mortar lining with spirally-wound reinforcing rod.

There are three methods of applying cement-mortar linings to pipelines in place: centrifugal method, reinforced centrifugal, and mandrel process. All three methods require thorough cleaning and dewatering.

Centrifugal Process.—A variable speed winch pulls a revolving mortar dispensing head through the pipe at a specified rate to control lining thickness. Excavation of access openings are required from 500–700 ft (150–200 m), depending on the diameters. Large diameter pipes can reduce this requirement when working areas inside the pipe become greater. In addition to the above, electric generators

are used to supply power to the winch and the mortar mixer. In sizes above 10 in. (250 mm), troweling is usually done to provide a smoother finish and ultimately a greater carrying capacity. In large sewers, service laterals are usually plugged prior to lining.

The cost of centrifugal in-place lining depends on a number of standard factors including pipe diameter, pipe length, condition of the line, depth and profile, access, traffic control, and thickness requirement. The greater the length to be lined usually reduces the unit costs significantly. Centrifugal in-place lining is applicable to pipe sizes from 8 in. (200 mm) to 144 in. (3,660 mm). The current "state-of-the-art" for this method can be most economical above 24 in. (610 mm) in diameter. One of the advantages to this cement lining method is that the line can be placed in service 24 hr after the lining process. This process is not a speedy one as compared to other renewal methods, but it has proven to be effective and in many cases competitive.

Reinforced Centrifugal Lining.—Severly deteriorated sewer pipes, above 24 in. (600 mm) diam may require additional reinforcement based upon structural design considerations. This reinforcing process consists of three steps. A course of mortar one-half the final lining thickness is placed by the centrifugal machine without troweling. Then a spirally wound reinforcing rod is put in place. The size and spacing of the rod are determined by design. After the steel rod has been placed, a second course of mortar is spun into place to the final desired thickness. The spiral rod has two advantages over prefabricated cage steel, i.e., it requires less steel, and it conforms to the inside contour of the pipeline. The addition of reinforcing and increased time raises the cost of this method significantly; however, when compared to gunite and reinforced concrete, it is competitive.

Mandrel Process.—The mandrel process, invented by an Australian named Tate, is applicable to sewer pipes from 4 in. (100 mm)–16 in. (410 mm) diam with few service connections. After the line is thoroughly cleaned and dewatered, the mandrel process can be accomplished. This process uses a pressurized extrusion technique. A tight-fitting baffle is threaded onto a cable in the pipeline, and a two-part sand and one-part cement mortar mix is loaded into the pipe against the baffle. A conically-shaped mandrel is then secured to the cable. The mandrel is composed of centering springs, metering springs, a perforated skirt, and a troweling edge.

The assembly is pulled through the pipe usually for 350 ft (100 m)–450 ft (150 m). Back pressure of the baffle forces the mortar over the cone of the mandrel and against the pipe wall. The perforated skirt squeezes out excess moisture, and the troweling edge produces a smooth, dense coating. Because of the pressure created during the lining process, service connections must be plugged.

This method has found minimal application in the sewer rehabilitation field due to frequent excavations required, slow speed, high cost, and diameter limitations.

Like other cement-based methods, their use is not recommended for rehabilitation where high sewage temperatures and acidic corrosion are constantly prevalent. For all cement lining projects, the sewer must be thoroughly cleaned and dewatered.

Reinforced Gunite Placement.—Large diameter deteriorated sewers can be

rehabilitated by the *Gunite* process, which is sand, cement, and water placed under pressure. Renewal by this process can be for structural purposes as well as interior scouring and erosion. In the case of aggressive sewage, special aggregates and high alumina cements may be used. Welded wire mat or small diameter rod reinforcing is used for structural gunite applications.

Premixed sand and cement in a dry state moves through a material hose under pressure to the nozzle. Another hose carries water to the nozzle at a higher pressure than that in the material base. Water is fed through a water ring into the dry material stream as it enters the nozzle, and here hydration of the cement starts. The material, mixed only with enough water to properly place it, continues through the nozzle to the place of deposit. The control of the proper amount of water is extremely important, and the application of gunite is greatly dependent upon operator skill and experience.

Guniting can be placed under low sewage flows; however, it is best to dewater the sewer totally. Compared to hand-placed mortar or machine-placed concrete, gunite is denser material with higher ultimate compressive strength. The percentage of voids is less than half of concrete, and a low permeability rate is obtained. It adheres well to other concrete and brick sewers and is more corrosion-resistant than normal concrete. This method is ideally suited for extremely deteriorated large sewers where men and equipment can work without restriction. Comparatively, this method is slower to install than cement mortar linings, but is economically feasible. The finish, when troweled, is similar in smoothness to cement mortar linings and will improve hydraulic characteristics. Long lengths of sewers may be effectively renewed with few excavations, and minimum traffic disruption. The greater the structural deterioration, the more effective the gunite process is compared to cement mortar linings.

Concrete Placement (Slipform or Fixed Form).—The placement of reinforced or nonreinforced concrete for rehabilitation is an effective method of renewing a variety of sewer conduit shapes. Normally, high sulfate-resistant portland cement is used in large diameter sewers where labor and materials can be handled effectively. In large sewer pipe and tunnels, hand-built forms or prefabricated forms are built or installed in the sewer, or both, and concrete is poured around them. The forms are removed and reused over and over until the entire sewer renewal is completed. The structural condition of the existing conduit determines whether or not steel reinforcement is required. Reinforcing steel can be single or multiple layers of welded wire mesh or hand-laid-up cages attached to the deteriorated conduit wall by threaded inserts, similar to the "Phillips Read Head" type.

One notable installation of this type is the Davern Street Storm Sewer in St. Paul, Minn. A horseshoe sewer tunnel was rehabilitated by inserting 16 ft (4.9 m) long prefabricated corrugated aluminum sleeves into the tunnel and pouring concrete around them. This proved to be less costly than total replacement, complete clay tile replacement, or other methods investigated.

Structurally, reinforced concrete lining rehabilitation is very credible and simple to design around. The sewer must be thoroughly cleaned and dewatered prior to rehabilitation. In many rehabilitation cases, concrete placement is the only solution. Seldom is it applicable below 48 in. (1,200 mm). It does, however, have renewal applications in virtually any shaped sewer conduit.

Polyvinylchloride Sheet Liners (PVC).—For additional corrosion protection

against aggressive sewage when reinforced or nonreinforced concrete is placed, protection can be obtained by the use of mechanically bonded PVC sheet liners. These liners contain interlocking ribs or T's to hold them in place while the concrete is curing. These linings are placed on the forms prior to pouring, and heat fusion is used to join each section of liner. After the forms are removed, the next section is joined by heat fusing at the joint. The entire PVC plastic liner is tested with a spark device to check for holes, defects, and damages. Hydraulic capacities are greatly improved, and corrosion of the concrete is virtually eliminated. The cost of the pliable plastic liners is reasonable; however, their installation is expensive but extremely effective if installed properly.

Fiberglass Reinforced Cement Liners (Segmented).—Fiberglass reinforced cement liners are thin panels developed to rehabilitate sewers above 42 in. (1,100 mm). These linings are segmented to fit the diameter required. Circular, oval, rectangular, elliptical, ovoid, or V-shaped sewers can be lined with these linings.

The linings are manufactured in a composite of cement and glass fibers. The normal wall thickness is 3/8 in. (10 mm); however, this is occasionally modified. The linings have high mechanical and impact strengths with good acid and alkalinity resistance. They are also highly resistant to abrasion, with negligible absorption and permeability.

After a thorough line cleaning and dewatering, the segments are installed in 4 ft (1.2 m) lengths, and they overlap at the joints. The flanges may be predrilled for fixing by screw or impact nail gun. The segmented rings are located by wedges, and, upon final assembly, the entire section installed is cement pressure grouted. Side laterals are cut in and grouted prior to final erection.

This method provides flexibility to accommodate variations in grade curvature, cross section, and deterioration. The linings are not designed to support structurally imposed earth loads; however, they are effective in providing new lining for a structurally sound sewer. The interior smooth surface improves hydraulic capabilities.

Although the segmented sections are light and easy to handle, the installation is labor intensive and slow. It is not a recommended method for structurally inadequate sewers. Invented in England, these linings are available on a custom designed basis in the United States. At this time, it is difficult to determine the competitive comparability of glass reinforced cement linings.

Fiberglass Reinforced Plastic Liners (Segmented).—Similar in nature to glass reinforced cement liners, these liners are used predominantly in sewers above 42 in. (1,100 mm). Formed in segments with varying wall thickness, these linings are not structural components, but they do provide an adequate corrosive barrier and smooth lining for structurally sound, but deteriorated, sewers.

These segmented liners are manufactured from fiberglass and a variety of resins from polyester to vinylester. Choices of resins can be made relative to the desired corrosion resistance required. This composite of fiberglass and resin provides excellent hydraulic characteristics due to the smooth inner surface. High mechanical and impact strengths are apparent, and resistance to abrasion is good. The linings offer little absorption and no apparent permeability. Wall thickness and designs can be varied easily for custom installations.

The installation of segmented glass reinforced plastic (GRP) liners is not difficult due to the light weight of the segments. After a thorough cleaning and dewatering,

the segments are placed in the sewer with mechanically locking joints, bolts, or impact fasteners at each joining section. They are blocked in place, and, after the entire section is complete, the total area between the linings and the existing sewer is cement pressure grouted in place to prevent sagging and deformation.

Excavation and Replacement

This method involves the removal of the existing pipes from the ground and replacing them with new ones. When suitable materials and construction methods are utilized, this technique *may* produce the most effective rehabilitation results. The cost of this technique, however, is normally much higher than other rehabilitation methods, and the time requirement is usually much longer.

Replacement is normally considered for application under one or more of the following conditions:

1. In locations where pipes have lost their structural integrity and are collapsed, crushed, broken, or badly deteriorated and cracked.
2. In cases where pipe size enlargement, change in grade, or pipe realinement are needed in addition to pipe deficiency corrections.
3. In cases where the causes of damages to the existing pipes (e.g., corrosion, soil movement, increasing traffic load,) have been identified, and it is desirable to prevent the reoccurence of these damages by replacing the existing pipes with new ones having higher quality and greater strength.

Just as for new sewer construction, this rehabilitation method may require the removal of pavement, disruption of traffic, dewatering, well-pointing, shoring, interference with utilities and structures, and repavement. In addition, during

TABLE 2.—Correction Methods

A. External
   1. Chemical grouting
   2. Cement grouting
B. Internal
   1. Chemical grouting
   2. Polyester resin lining
   3. Sliplining—polyethylene (ext.) PE
   4. Sliplining—polyethylene (spiral weld) PE
   5. Sliplining—polybutylene (ext.) PB
   6. Sliplining—reinforced plastic mortar pipe—RPMP
   7. Sliplining—reinforced thermosetting resin pipe—RTRP
   8. Cement mortar lining placement
   9. Cement mortar lining, reinforced
   10. Cement mortar lining, mandrel process
   11. Reinforced gunite placement
   12. Concrete placement
   13. Concrete placement with PVC sheet liners
   14. Glass reinforced cement liners (segmented)
   15. Glass reinforced plastic liners (segmented)
   16. Excavation and replacement or repair

TABLE 3.—Selection Chart: Sanitary Sewer Pipeline Rehabilitation Methods

(1)	Problem description (2)	Possible correction (3)	Dewatering requirements (4)	Cleaning requirements (5)	Remarks (6)
1	High ground-water flow in pipe zone	A-1	NA	NA	Check fluctuation and backfill soil conditions
2	External voids adj. to pipe		NA	NA	Careful control required of material mix selection important
	a. small	A-1	NA	NA	
	b. large	A-2	NA	NA	
	Small diameter 6 in. (150 mm)–9 in. (230 mm)				
3	Deteriorated pipe joints	B-1,2	Complete dewatering	Thorough cleaning	
4	Leaking offset joints	B-1, 2, 3, 16	Complete dewatering	Depends on method	Check angular offsets
5	Open joints & sagging	B-1, 2, 3, 16	Complete dewatering	Thorough cleaning	Check ground-water movement and levels
6	Cracked pipe	B-2, 3, 10, 16	Complete dewatering	Thorough cleaning	Determine structural condition
7	Broken or crushed pipe	B-3, 16	Complete dewatering	Thorough cleaning	If using B-3, check annular space grouting
8	Leaking service connections	A-1, B-1, 3, B-16	Complete dewatering	Thorough cleaning	Check ground-water movement
9	Collapsed pipe	B-3, 16	Complete dewatering	Thorough cleaning	Determine soil settlement and damage extent
10	Loss of pipe zone support	A-2, B-16	Complete dewatering	Thorough cleaning	Use A-2 if pipe has not sagged
11	Faulty taps between service connections & main sewer	B-1, 16	Complete dewatering	Not required	B-1 is not structural
12	Crown or invert corrosion	B-2, 3, 10	Complete dewatering	Thorough cleaning	Check structural condition
	Lg. Diameter 8 in. (457.2 mm) and up				
13	Deteriorated pipe joints	B-1, 3	Complete dewatering	Thorough cleaning	B-1 is manual nozzle type, sliplining if excessive
14	Leaking offset joints	B-1, 2*, 3, 6	Some dewatering	Thorough cleaning	*B-2 to 30 in. (760 mm), check angular offsets
15	Open joints and sagging	B-1*, 3, 4, 6, 7	Some dewatering	Thorough cleaning	Fill annular space if high ground-water movement

TABLE 3.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
16	Cracked pipe and slab outs	B-3*, 4, 6, 7, 8, 9, 11, 12, 14, 14	Check method	Depends on method	Cement grout annular space if structurally unsound
17	Collapsed pipe	B-3, 6, 7, 16	Check method	Thorough cleaning	Remove collapsed portion; cement grout if needed
18	Leaking connections	A-1, B-1, 16	Flow reduction	Not required	In large pipe B-16 usually not needed
19	Loss of pipe zone support	A-2, B-16	Check method	Depends on method	Use A-2 if pipe has not sagged
20	Crown or invert corrosion	B-3, 4, 6, 7, 8, 9, 10, 11, 14, 15	Check method	Clean if invert erroded	
21	Loss of tile liners or	B-3, 4, 6, 7, 11, 12, 14, 15, 16	Check method	Thorough cleaning	Remove all loose tile or brick; sandblast if needed
22	Faulty taps and manhole connections	B-12, 16	Flow reduction	Not required	In some cases B-12 will work inside large diameter
23	Rebar corrosion	B-3, 4, 6, 7, 8, 9, 11, 12, 14, 15	Check method	Depends on method	Check structural condition

the construction period, the sewage flows in the various sewer sections must be bypassed. All the costs involved in these tasks should be considered when comparing this technique with other rehabilitation techniques.

Conclusions

It is extremely important to evaluate the physical condition of the existing sewer to adequately determine the best rehabilitation method and the most apparent cost-effective solution.

In most cases, the rehabilitation of sewer pipelines can be accomplished effectively and at a lower cost than excavation and replacement (see Table 2).

Many unique materials and techniques are available for sewer pipeline rehabilitation, and consideration should be given to external problems as well as internal problems. The diameter, cross section, variation in flow, number of connections, dewatering requirements, access, location, depth and nature of the effluent, movement of the ground water, amount of extraneous flow, and soil settlement conditions are all additional factors of rehabilitation evaluation concern.

External grouting and void filling help to solve ground-water infiltration, sewage exfiltration, and settlement problems. Chemical grouting, sliplining with various pipes, and placement of several mechanical and cementatious materials are the basic known techniques currently utilized in sewer pipeline rehabilitation. Ongoing

research and testing of new techniques and material will provide the rehabilitation process with up-to-date and economical solutions (see Table 3).

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DISCUSSION

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CRACKING IN CONCRETE PAVEMENTS[a]

Closure by Robert C. Deen,[6] James H. Havens,[7] Assaf S. Rahal,[8] and W. Vernon Azevedo[9]

Laughter's perception probably goes far beyond the point of his questions. The drag theory provides a basis for spacing expansion joints. Forces may be visualized as transient or standing waves of uniform or varying amplitude along the longitudinal axis of a pavement. Some sections may be in the sun while others may be shaded at a given time. To understand the mechanics, visualize a midsection of pavement held fixed by gravity. When the temperature rises, the ends are unconstrained, the resistance will be due to drag only. If the compressive strength is 5,000 psi (35 MPa) and if the coefficient of friction between the pavement and foundation is unity, it is uniquely determined that the compressive stress in the concrete at the midsection is 5,000 psi (35 MPa) when the length of pavement is 5,000 ft (1,500 m) in each direction [temperature rise of 100° F (38° C)]. Explosive failure (blowup) would occur—fatigue would occur in less critical situations. Now, visualize sawed contraction joints that reduce the bearing area 30%. Then the critical length of pavement is 3,500 ft (1,100 m) in each direction from the midsection when the compressive strength is 5,000 psi (35 MPa). Visualize further a relief (expansion) joint at the midsection resisting compression with a nearly constant back pressure of 2,000 psi (14 MPa). Surely this stress would not be damaging to 5,000-lb (35-MPa) concrete when bearing on the full cross section; however, the same force or thrust bearing on a sawed-type contraction joint would magnify the stress (1 ÷ 0.70 = 1.43 times) in this example to 2,857 psi (19.7 MPa), which surely would fatigue 3,500-pound (24.1-MPa) concrete. For sake of brevity, consider relief joints capable of exerting a restoring pressure of 2,000 psi (14 MPa) at 2,000-ft (610-m) intervals and no sawed joints; the midsection should not creep unless an imbalance arises exceeding the restoring pressure (exerted by the expansion joint filler) by at least 2,000 psi (14 MPa) (drag resistance). This force cannot be generated within the section or from the ends. Sections or slabs would not necessarily move downgrade nor would blowups concentrate at sags. This tendency has not been observed.

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[a] March, 1980, by Robert C. Deen, James H. Havens, Assaf S. Rahal, and W. Vernon Azevedo (Proc. Paper 15273).

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Bear in mind that the interval between tension cracks uniquely reflects the tensile strength of the concrete at the onset of the first critical shrinkage or temperature contraction. If there were no sawed (reduced) cross sections, the crack interval in feet would be equal to the tensile strength (stress) in pounds per square inch. If the cracking initiates randomly and an occasional interval slightly less than $2L$ occurs, an intervening crack will not occur.

Compression at joints (sawed) causes insidious fatigue and eventually becomes apparent as *D* cracking. Concrete at some joints may be stronger than at others. Generally, early signs of *D* cracking appear rather uniformly throughout. Compression would tend to be uniform. As fatigue and deterioration progress, the weaker locations crumble or blow up—thus relieving the stress at joints nearby. Even so, a regularity of interval evolves. The blowup or bump-type failure will be found at 1/2-mile–1-mile (0.8-km–1.6-km) intervals in pavements made of ordinary, good concrete. Certainly, some pavements endure longer than others before crumbling occurs. The stronger concretes tend to blow up very explosively.

Condition Evaluation of Jointed Concrete Airfield Pavements[a]

Discussion by Philip P. Brown,[4] F. ASCE, Michael P. Jones,[5] M. ASCE, and Robert B. Brownie[6]

Since 1969 the Naval Facilities Engineering Command has been using a quantified statistically based condition survey procedure for rating the airfield pavements at approximately 75 Naval and Marine Corps airfields throughout the world. These ratings have been used as a basis for establishing the need and priority for repair. The pavement condition index (PCI) procedures described by the authors have the same basic objectives and many of the techniques utilized in the Navy method. There are significant differences, however, in the weighting of defects which warrant discussion.

In the PCI procedure, "deduct values" were developed and adjusted to fit a curve of judgmental ratings by experienced engineers. The Navy developed "severity weights" based on an assessment of operational and maintenance

[a]July, 1980, by Mohamed Y. Shahin, Michael I. Darter, and Starr D. Kohn (Proc. Paper 15527).

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requirements. Severity weights of various pavement defects were developed from the following series of questions which were put to operational and maintenance personnel as well as to experienced pavement engineers.

1. Does the defect affect the safe operation of aircraft now?
2. Will the defect lead to increased airfield maintenance effort within 1 yr–2 yr?
3. Will the defect lead to increased aircraft operational costs within 1 yr–2 yr?
4. Will the defect result in significant deterioration of load carrying capacity within 5 yr?
5. Will the defect lead to unsafe aircraft operations within 5 yr?

These questions were paraphrased from criteria used as the basis for a comprehensive Navy maintenance management plan for all facilities. Of these five questions, a positive answer to the first was given twice the weight of the following four. Answers were tabulated separately for 11 types of defects in both concrete and asphalt pavements. The assigned weights were determined as the sum of scores for each defect type.

Another major difference in the Navy condition evaluation procedure is an adjustment in the severity weights for environmental factors. Severity weights are scaled down for an individual airfield which is not subject to a freezing climate, heavy rainfall, poor draining subgrade, or fine blow sand such as in a desert area. Each of these environmental factors has an effect on the significance of occurrence of various defects. For example, defective joint seal material is of greater concern in areas of heavy rainfall, or a poor draining subgrade, or in areas where blow sand can infiltrate joints. Similarly, when considering asphalt pavement defects, surface ravelling is more likely to progress in a freezing climate than in a temperate climate. Although the environmental modifiers are largely judgmental, their use has been found to be necessary in assigning repair priorities among airfields in various regions of the country.

The purpose of the Navy's approach was to improve on the earlier condition rating procedures which subjectively rated pavements as "good," "fair," "poor," etc. In the past, engineers have tended to rate pavements more on perceived structural defects than on functional defects. Some older rating systems considered a crack (transverse or longitudinal) in a concrete pavement to be a major defect, while surface spalling was minor. In regard to operational suitability, the latter defect can be much more serious. The PCI procedures appear to have taken this into account through the use of three severity level scales for each defect.

At present, neither the Air Force or Navy condition evaluation procedures can adequately account for the operational importance of one pavement area or airfield over another. Although it is possible to develop uniform and systematic condition ratings which provide a tool for allocating maintenance funds, it has not been possible to incorporate the airfield mission as a factor in the system. For example, individual Navy Air Commands frequently override condition survey priorities to maintain its major airfields in a superior state of repair. The Navy has plans to address this problem in a current program to develop an improved pavement maintenance management system.

CAPTIVITY AND CHOICE IN TRAVEL-BEHAVIOR MODELS[a]

Errata

The following corrections should be made to the original paper:

Page 430, paragraph 2, line 1: Should read "In another situation, captives are included in the calibration data." instead of "In another situation, captives, are included in the calibration data."
Page 433, Fig. 1: Caption should read "True and Biased MNL Models for Data With and Without Captives Included (Auto Captives Predominating—Auto Probabilities)" instead of "True and Biased MNL Models for Data With and Without Captives (Included: Auto Captives Predominating—Auto Probabilities)"
Page 433, Fig. 2: Caption should read "True and Biased MNL Models for Data With and Without Captives Included (Auto Captives Predominating—Bus Probabilities)" instead of "True and Biased MNL Models for Data With and Without Captives (Included: Auto Captives Predominating—Bus Probabilities)"

PAVEMENT ANALYSIS BY PROGRAMMABLE CALCULATORS[b]

Discussion by Stephen F. Brown[3] and Janet M. Brunton[4]

The authors are to be complimented on presenting procedures which make structural analysis and design methods for asphalt pavements more accessible to potential users. However, the writers are concerned about the accuracy and limitations of the method of equivalent thickness for pavement analysis and would also point out that there are certain errors in the text of the paper. They do so against the background of having performed for the authors the computations using the BISTRO (13) and PONOS (5) programs, which were used as the basis for assessing the accuracy of the more approximate calculations which they outlined in the paper.

The symbol in Eq. 8 is not the same as that appearing subsequently and defined in Appendix II as "number of layers." In Eq. 8 it should be defined

[a] July, 1980, by Peter R. Stopher (Proc. Paper 15566).
[b] September, 1980, by Per Ullidtz and Kenneth R. Peattie (Proc. Paper 15682).
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as

$$n = 0.83 \log \left(\frac{4 \times 10^4}{S_b} \right) \quad (23)$$

in which S_b = stiffness modulus of bitumen, in megapascals.

In Eq. 18 the exponent 2/3 should 3/2 giving the Boussinesq expression for vertical stress as

$$\sigma_z = \sigma_o \left\{ 1 - \frac{1}{\left[1 + \left(\frac{a}{z} \right)^2 \right]^{3/2}} \right\} \quad (24)$$

Eq. 14, which presents a general expression for the equivalent thickness of the top $(n - 1)$ layers in an n layer system, should be

$$h_{e,n} = \sum_{i=1}^{n-1} (f_{n-1} f_{n-2} \ldots f_i) h_i \sqrt[3]{\frac{E_i}{E_n}} \quad (25)$$

in which $f_1 = 1.0$ and f_2 to $f_{n-1} = 0.8$.

An exception to this is the case for a two-layer system when the single correction factor is given by the authors as 0.9, thus

$$h_{e,2} = 0.9\, h_1 \sqrt[3]{\frac{E_1}{E_2}} \quad (26)$$

Furthermore, if the different values of Poisson's ratio are known for the various layers, then the cube root term in Eq. 25 becomes $[E_i(1 - \nu_n^2)] / [E_n(1 - \nu_i^2)]$.

There are certain restrictions to the use of the MET which should be closely examined. The condition that no layer should be thinner than half the radius of the loaded area effectively eliminates analysis of problems with layers thinner than about 60 mm. Therefore, for instance, a separate asphalt wearing course could not be modelled.

The necessity to ensure that E_i/E_{i+1} is greater than two is a more serious restriction. This effectively means that all asphalt layers need to be combined into a single one for analysis, since wearing courses,ifor instance, are usually less stiff than bases. For an approximate procedure, it is quite reasonable to consider all the asphaltic layers together, but the authors have implied that their method may be used for a multilayered system. Most pavement structures can realistically be modelled by a two- or three-layered system and perhaps the MET should be restricted to these fairly simple cases.

Integration of the Boussinesq solution for a point load cannot readily be performed to deal with a uniformly distributed load except to provide solutions on the axis of symmetry. The authors, therefore, suggest that in such cases, for systems with a stiff top layer, the point load solution will usually give satisfactory results. They should give more specific guidance than this to potential users as the off-axis solution is of wide interest, particularly when dealing with dual wheel loads when stresses and strains midway between the loads are required.

The authors have presented data to indicate the accuracy of MET and other

simplified computations by comparing the results with those obtained using more accurate computer programs, viz PONOS for asphalt stiffness and either BISTRO or CHEVRON for pavement analysis. Further computations have been performed by the writers as part of an exercise of assessing the MET procedures.

The writers used a wide range of values for elastic modulus and layer thickness in a series of realistic pavement structures. The results of computations using MET were compared with those from BISTRO in a similar way to Table 8 in the paper. Comparisons are presented in Table 10. The means ($\bar{x}$) and standard deviations (s) are of the ratio: Strain computed by MET/Strain computed by BISTRO.

The writers' results indicate that the accuracy of the MET is much less

TABLE 10.—Comparisons of Strain Computations by MET and BISTRO

Value	New Computations			Authors: Table 8	
	Number of comparisons	$\bar{x}$	s	$\bar{x}$	s
(1)	(2)	(3)	(4)	(5)	(6)
ϵ_z center	17	1.39	0.27	1.03	0.14
ϵ_t center	17	1.01	0.22	1.05	0.24
ϵ_z off-center	22	1.29	0.08	—	—
ϵ_t off-center	22	0.94	0.25	—	—
Both off-center	44	1.12	0.25	1.05	0.14
All values	78	1.14	0.28	1.04	0.16

TABLE 11.—Comparison of Pavement Lives Based on MET with Those Based on BISTRO

Value (1)	$\bar{x}$ (2)	s (3)	Range of $\epsilon_{MET}/\epsilon_{BISTRO}$ $\bar{x} \pm s$ (4)	Range of N_{MET}/N_{BISTRO} (5)
ϵ_z center	1.03	0.14	0.89–1.17	1.52–0.57
ϵ_t center	1.05	0.24	0.81–1.29	2.75–0.29[a]
				1.93–0.45[b]

[a]Based on Eq. 28.
[b]Based on Eq. 29.

impressive than was indicated in the paper. This points to the need for a more careful and exhaustive check of MET results over a wide range of conditions, so that users are aware of the accuracy of the results they may obtain.

Another way of assessing the reliability of MET results is to examine the effect of the discrepancies with BISTRO in terms of potential pavement life. The subgrade strain can be used to estimate life in terms of standard axles (N) to give a critical rut depth of 20 mm using the equation

$$N = \frac{3.02 \times 10^{15}}{\epsilon_z^{3.57}} \qquad (27)$$

in which ϵ_z is in microstrain.

Similarly, maximum tensile strain in the asphalt layer may be used to estimate fatigue life, the particular expression depending on the actual asphalt mix involved. For a typical gap graded material

$$N = \frac{1.22 \times 10^{17}}{\epsilon_t^{4.81}} \qquad (28)$$

while for a typical continuously graded mix

$$N = \frac{2.12 \times 10^{12}}{\epsilon_t^{3.11}} \qquad (29)$$

Eqs. 27–29 are taken from Brown (18). Making use of the data in Table 8 of the paper, the writers compiled Table 11 to indicate the ratios of lives calculated by MET strains to those based on BISTRO strains. The rather large range of lives indicated in Col. 5 of Table 11 again calls into question the accuracy of the MET analysis method.

Appendix.—Reference

18. Brown, S. F., "An Introduction to the Analytical Design of Bituminous Pavements," University of Nottingham, Nottingham, England, 1980.

TECHNICAL PAPERS

Original papers should be submitted in triplicate to the Manager of Technical and Professional Publications, ASCE, 345 East 47th Street, New York, N.Y. 10017. Authors must indicate the Technical Division or Council, Technical Committee, Subcommittee, and Task Committee (if any) to which the paper should be referred. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles must have a length not exceeding 50 characters and spaces.

2. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8-1/2-in. (220-mm) by 11-in. (280-mm) paper. Three copies of all figures and tables must be included.

3. Generally, the maximum length of a paper is 10,000 word-equivalents. As an *approximation*, each full manuscript page of text, tables or figures is the equivalent of 300 words. If a particular subject cannot be adequately presented within the 10,000-word limit, the paper should be accompanied by a rationale for the overlength. This will permit rapid review and approval by the Division or Council Publications and Executive Committees and the Society's Committee on Publications. Valuable contributions to the Society's publications are not intended to be discouraged by this procedure.

4. The author's full name, Society membership grade, and a footnote stating present employment must appear on the first page of the paper. Authors need not be Society members.

5. All mathematics must be typewritten and special symbols must be identified properly. The letter symbols used should be defined where they first appear, in figures, tables, or text, and arranged alphabetically in an appendix at the end of the paper titled Appendix.—Notation.

6. Standard definitions and symbols should be used. Reference should be made to the lists published by the American National Standards Institute and to the *Authors' Guide to the Publications of ASCE*.

7. Figures should be drawn in black ink, at a size that, with a 50% reduction, would have a published width in the *Journals* of from 3 in. (76 mm) to 4-1/2 in. (110 mm). The lettering must be legible at the reduced size. Photographs should be submitted as glossy prints. Explanations and descriptions must be placed in text rather than within the figure.

8. Tables should be typed (an original ribbon copy and two duplicates) on one side of 8-1/2-in. (220-mm) by 11-in. (280-mm) paper. An explanation of each table must appear in the text.

9. References cited in text should be arranged in alphabetical order in an appendix at the end of the paper, or preceding the Appendix.—Notation, as an Appendix.—References.

10. A list of key words and an information retrieval abstract of 175 words should be provided with each paper.

11. A summary of approximately 40 words must accompany the paper.

12. A set of conclusions must end the paper.

13. Dual units, i.e., U.S. Customary followed by SI (International System) units in parentheses, should be used throughout the paper.

14. A practical applications section should be included also, if appropriate.